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4. The author's full name, Society membership grade, and footnote reference stating present employment must appear on the first page of the paper.

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9. Papers must be written in the third person.

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Journal of the
HYDRAULICS DIVISION
roceedings of the American Society of Civil Engineers

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DISCUSSION

(over)

Journal of the
HYDRAULICS DIVISION
Proceedings of the American Society of Civil Engineers

FLOOD DISTRIBUTION PROBLEMS BELOW OLD RIVER

By Frederic M. Chatry[1]

SYNOPSIS

suring the safe passage of flood flows on the Mississippi River at and Old River requires that a controlled system of natural streams and ar-l floodways be operated to distribute peak flows in accordance with the 1 capacities of the various system segments. Factors influencing the op-n include stages and discharges, sedimentation problems, economic and ercial considerations, and adaptation to constantly changing hydraulic ions.

INTRODUCTION

the limits of the watershed tributary to the Mississippi River are drawn ordinary map, the figure inscribed can be visualized as a large, upright , with its mouth stretching across much of the northern United States, s spout in Louisiana. In this funnel the runoff is collected from a drain-sin 1,246,000 sq miles in area, in which are contained all or parts of 31 states of the United States, and 2 Canadian provinces. Through the fun-spout pour 468,000,000 acre-ft of water in an average year, at rates g from 100,000 cfs during low water season, to nearly 2,000,000 cfs in a l high water, and to 3,000,000 cfs for the design flood.

en the Flood Control Act of 1928 projected the federal government into eld of flood control in a preemptive manner, the mechanism which the

te.—Discussion open until January 1, 1961. To extend the closing date one month, en request must be filed with the Executive Secretary, ASCE. This paper is part opyrighted Journal of the Hydraulics Division, Proceedings of the American So-f Civil Engineers, Vol. 86, No. HY 8, August, 1960.

1 gr., U. S. Army Engr. Dist. New Orleans, New Orleans, La.

end of our funnel represents was relatively simple. Flow entered the via the Mississippi and Red Rivers, which streams are connected togeth a short length of channel called Old River, and was then conveyed to the of Mexico by way of the Mississippi River, and the Atchafalaya River. E for levees, neither outlet was subject to any control, and flood manage was a matter of constructing and maintaining the levee system to suffi height and cross-section to insure its integrity. However, this was not e done, and levee overtoppings and failures were experienced in nearly e significant high water. Before embarking on a description of the control tem that grew out of the Flood Control Act of 1928, some remarks on the eral topography and hydrography at and below the latitude of Old River a order. Present details of the area and of the flood control plan are show Fig. 1. Fig. 2 shows the area prior to start of construction of the flood trol plan.

NATURAL SEGMENTS OF THE FLOOD CARRYING SYSTEM AT AND BELOW OLD RIVER

The largest segment of the natural complex at and below Old River is Mississippi. The Mississippi River adopted its present course below Old F approximately 1200 A.D., after abandoning the Teche and Lafourche distr taries. The present distance from Old River to the Gulf, via Southwest F is 321 miles. Below the gap at Old River, the Mississippi is fenced off t west by a continuous levee extending almost to the river's mouth. On the bank, the river is confined by high bluffs as far south as Baton Rouge, ar a continuous levee thereafter. The carrying capacity of the leveed cha varies throughout its length. Just above Old River there is capacity 2,700,000 cfs. This figure decreases to 2,100,000 cfs below Old River, to proximately 1,550,000 cfs below Morganza Floodway, and finally to 1,250 cfs below Bonnet Carre Spillway. The Mississippi River disposes of slig less than 70% of the total annual flow arriving at the latitude of Old River. though this percentage has been decreasing since 1855, exhaustive stu made in connection with the Old River control project do not reveal any nificant corresponding decrease in carrying capacity. As a matter of fact flow capacity of the River below Old River, as demonstrated by the st discharge relationship at that point, has been subject to no progressive ch at all, but has rather been subject to both increases and decreases in flow pacity. In 1950, for example, the stage-discharge relationship was as fa able as it has ever been throughout the period for which measurements available.

The second segment, with regard to size, is the Atchafalaya River. stream probably was spawned by excess water from the Mississippi R spilling over the ridges of that stream's abandoned Lafourche and T courses. This excess water flowed into the low area, which is now called Atchafalaya Basin and, in the process, developed a large number of small tributary basins, which ultimately joined to form the Atchafalaya River. sequent development of the channel was retarded by the formation of an tensive dam of drift and debris called a "raft." This raft at its point of m mum development extended downstream for approximately 30 mi and stret from bank to bank. Pressures generated by the need for a navigation ro

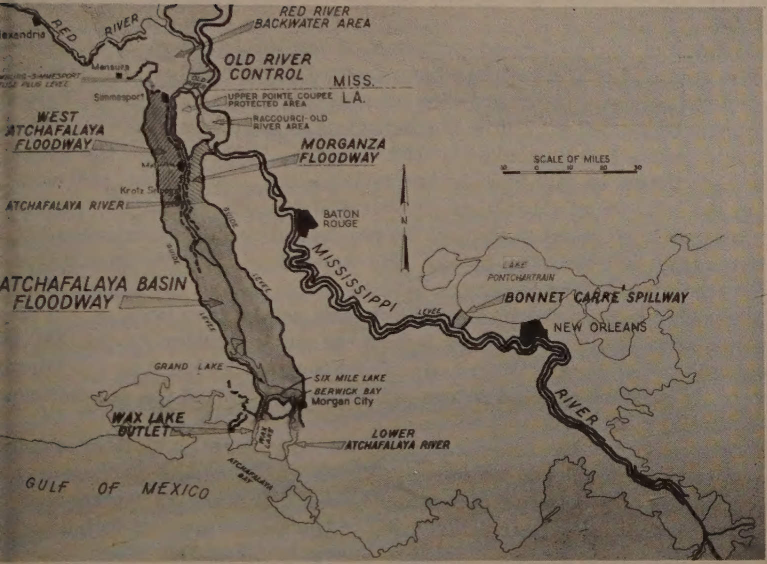


FIG. 1.—FLOOD CONTROL PLAN BELOW OLD RIVER

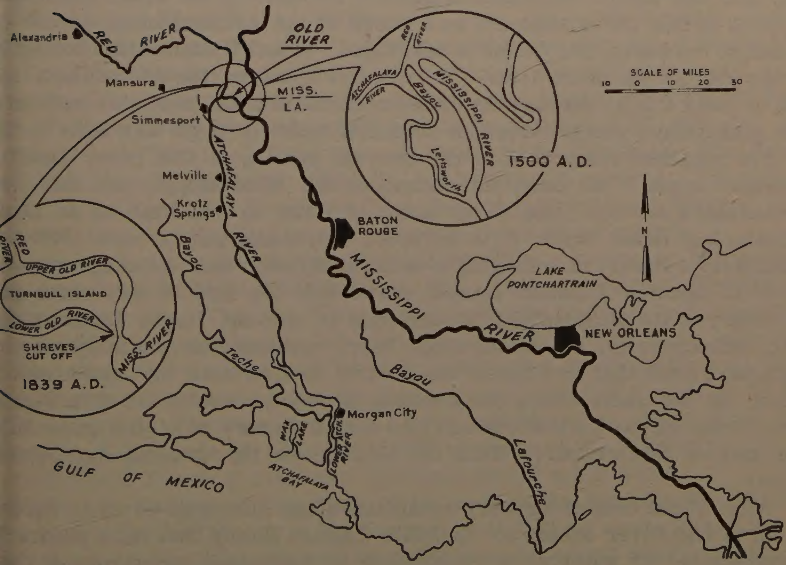


FIG. 2.—ALLUVIAL AND DELTAIC PLANES

combined with overflow of cleared lands above the raft led to its removal dertaken in 1839. The removal was completed just prior to the War bet the States.

Enlargement of the leveed portions of the Atchafalaya River during after the removal of the raft has been dramatically swift. At the time that moval of the raft was undertaken the normal discharge capacity at Simmes La. (approximately 5 mi below the Atchafalaya head) was only 30,000 cfs 1858, this figure had increased to 130,000 cfs. At the turn of the century figure had grown to 260,000 cfs and in 1950, the year of the last signifi flood at the latitude of Old River, it was 470,000 cfs. Even though there been no significant flood in the last 10 yr, the indicated normal capacity is approximately 500,000 cfs. It was this record of growth, in addition to fact that the Atchafalaya River gradient is double that of the Mississippi, w threw into sharp focus the imminence of what would have been an act of pi of monumental proportions, and led to the authorization of the $70,00C project called Old River Control to prevent the abandonment by the Mississ River of its present channel below Old River in favor of the steeper rout the Atchafalaya River.

Unfortunately for the American taxpayer in general, and flood control gineers in particular, the unleveed central reach of the Atchafalaya R which consists of a network of braided streams, many of which have a ments contrary to the general direction of the main flow, exhibited no ten cy toward self-enlargement. Instead, this portion of the river has con ously deteriorated through extensive siltation. The result of this siltatio been doubly detrimental to flow, not only reducing the available cross-sec but converting vast open water areas of low roughness to vegetated land a of high roughness.

The Atchafalaya River terminates in a deep, well-defined outlet via Ber Bay and the Lower Atchafalaya River. Because of the small quantity o carried in this lower segment as a result of the sedimentation in the ce reach of the basin, this outlet is maintained in an efficient condition by th gress and egress of the tides. Eventually, however, the Atchafalaya wil gin to build a delta through its outlet section. In addition to this natural o the Atchafalaya was provided an artificial outlet through Wax Lake in 19

The smallest component of the overall system is Old River itself. stream, some 7 mi long, interconnects the Mississippi, the Red, and Atchafalaya Rivers. Old River actually flows in one limb of an aband Mississippi River bend. It is believed that sometime around 1500 A.D Mississippi River, in enlarging a westwardly migrating bend around wha later to become Turnbull Island, caved into the Red River, which was tributary to the Mississippi through what is now the Bayou Lettsworth nel. Subsequently, the Mississippi River continued its westerly meande with the result that Red River was forced to flow into the upper side o bend. At the same time, the junction probably resulted in diversion of away from Bayou Lettsworth and into a distributary near the present s the city of Simmesport. This distributary is the present day Atchaf River.

After these conditions were established, the climactic event in the dev ment of Old River occurred. In 1831, Captain Henry Shreve, a pioneer i development of waterborne commerce in Louisiana, constructed the which bears his name. Shreve's cutoff severed the migrating bend previ referred to and formed Turnbull Island. The upper portion of the bend

nown as Upper Old River, whereas the lower portion was called Lower ver. Shreve's cutoff also halted the growth of the Atchafalaya River Jpper Old River no longer exists, except as a series of swales. Though elopment of Old River lagged that of the Atchafalaya for many years, it developing at a rate comparable to that of the Atchafalaya River. It is le that Old River is still inhibiting the growth of the Atchafalaya, inas- as it must continue to enlarge to meet the demands for water imposed Atchafalaya River at all stages.

final segment of the natural system is the vast reservoir known as the ver Backwater Area. This is the area subject to inundation by Mississ- ver backwater through the gap in the levee system at Old River. Its embrace a total of 2,700 sq mi, and, at the elevation of the project de- ood, it is capable of storing 20,000,000 acre-ft of floodwaters. Because ubject to frequent overflow, the entire area is sparsely developed and inhabited. There is, however, a significant aggregate of improved lands , some of which are now partially protected by levees, which makes the factor to be considered in distributing major floods.

OVERALL SYSTEM FOR CONTROL OF FLOODS AT AND BELOW OLD RIVER

ddition to the natural segments, the overall project includes several al segments that will be used in combination with the natural segments ribute flood flows properly throughout the system.

Atchafalaya Basin Floodway was formed by constructing "guide" levees d west of the Atchafalaya River, and generally parallel to the alignment stream. On the east side, the levee is carried upstream to approxi- the latitude of Melville, La., and then brought northeastward to join e Mississippi River levee. On the west side, the levee extends above itude of Old River and ties to high ground in the vicinity of Mansura, ie floodway thus created is intended to provide one-half of the total ca- required to dispose of the project design flood. During such a flood the te of inflow to the floodway would rise to as much as 1,500,000 cfs. In to introduce this total flow, three intakes will be employed. The first e is the Atchafalaya River, which is not subject to control. The other e artificial overbank floodways that flank the Atchafalaya River to the d west, both of which are subject to some measure of control.

floodway to the east of the Atchafalaya River, known as Morganza ay, receives its flow directly from the Mississippi River through a ontrol structure. The structure is of reinforced concrete and consists eir 3,906 ft long equipped with vertical lift gates. The weir, with its t elevation 37.5 ft mean sea level is subdivided horizontally into 125 ach of which provides a clear opening approximately 28 ft wide by 25 ft The Morganza Floodway extends as far southward as the latitude of Springs, La., where it will deliver its outflows to the floodplain of the Atchafalaya Basin. It averages approximately 5 mi in width.

intake for Morganza Floodway is located on the Mississippi River 7 miles below Old River, in the Raccourci-Old River area. Raccourci- ver is an ox-bow lake, formed by the Raccourci cutoff which occurred . The lake is expected to materially assist ingress of floodwaters into dway, and is further expected to act as a natural silt basin after the y is placed in use.

The Morganza Floodway is designated to divert approximately 600,00 from the Mississippi River to the Atchafalaya Basin. The lift gates pr complete flexibility in operation, in that flow through the floodway ca started, stopped, or adjusted as required. The Morganza Floodway was pleted in 1956, and has not been used since completion. An overall view c control structure is shown in Fig. 3. Details of the weir and gates can be in Fig. 4.

On the western side of the Atchafalaya River lies the West Atchafa Floodway, larger in area than the Morganza Floodway and potentially n more controversial in operation. The West Atchafalaya Floodway differs f the Morganza Floodway in that control is not achieved by the positive me nism of gates, but by a levee of substandard grade called a fuseplug levee theory, the grade of such a levee is set to permit overtopping when the f in progress reaches the point where operation of the floodway is require obtain the desired flow distribution. After overtopping, natural erosion of levee, augmented as required by artificial degrading will be relied upon to sure the required diversion.

The West Atchafalaya Floodway extends southward to the latitude of K Springs where, like the Morganza Floodway, it will discharge its floodwa to the central Atchafalaya Basin Floodway. It averages approximately 8 m width. Unlike Morganza Floodway, which is practically uninhabited, the W Atchafalaya Floodway supports a population of approximately 3,000 and c tains numerous improvements and much livestock.

The West Atchafalaya Floodway has not been used, and will only come operation for a flood which approaches the project design value in magnit Fig. 5 shows the fuseplug levee at the head of the floodway.

The final component of the Atchafalaya Basin Floodway is the Wax L Outlet into the Gulf of Mexico. This auxiliary outlet was constructed in 1 by dredging a channel from Six Mile Lake, through the Bayou Teche Ridge Atchafalaya Bay. The auxiliary outlet accomodates approximately 20% of total Atchafalaya Basin flow. An aerial view of the outlet is shown in Fig

The third artificial floodway in the overall plan is Bonnet Carre Spillw which is located on the east bank of the Mississippi River 33 miles above N Orleans. It was constructed to divert excess Mississippi floodwaters Lake Pontchartrain. It is the oldest of the artificial floodways, having b completed in 1936. The control structure, which is located at the river en the Floodway, consists of a reinforced concrete weir, 7,698 ft in length, s divided into 350 bays, 176 of which have a crest elevation of 15.8 ft msl, 174 of which have a crest elevation of 17.8 ft msl. Control is by wooden t bers or "needles" which measure 8 in by 12 in. The Spillway was designe discharge a peak flow of 250,000 cfs at a stage of 20 ft at New Orleans. It been operated in 1937, 1945, and 1950. In 1945, the design discharge equaled or exceeded on 15 days.

Significant problems have been encountered in the operation of Bon Carre Spillway, in that sedimentation in both the forebay area (the area tween the river bank and the control weir) and the Spillway proper are op ating to reduce its capacity in a progressive manner. Although the flow ca city can be maintained indefinitely by removing the silt deposits, such ma tenance is very expensive. View of the Spillway in operation are shown Figs. 7 and 8.

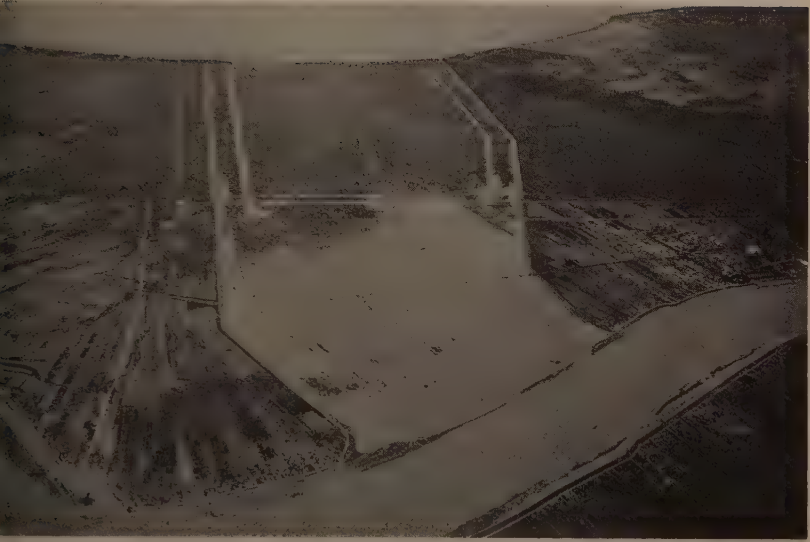
Although it is not yet a functioning segment of the overall plan, the River Control project is sufficiently close to completion that consideration it should properly be included in any description of the overall system.

. 3.—CONTROL STRUCTURE OF MORGANZA FLOODWAY. MISSISSIPPI RIVER VISIBLE IN THE BACKGROUND TO THE LEFT.

FIG. 4.—MORGANZA FLOODWAY. CLOSE-UP VIEW OF CONTROL STRUCTURE, FROM FOREBAY.

FIG. 5.—WEST ATCHAFALAYA FLOODWAY. SIMMESPORT-HAMBURG LEVEE AT HEAD OF THE FLOODWAY.

FIG. 6.—WAX LAKE OUTLET. VIEWED IN THE DIRECTION OF FLOW.



7.—BONNET CARRE SPILLWAY. VIEW DURING 1950 OPERATION. MISSISSIPPI RIVER IN FOREGROUND. LAKE PONTCHARTRAIN IN BACKGROUND.

. 8.—BONNET CARRE SPILLWAY. CLOSE-UP OF CONTROL STRUCTURE 1950 OPERATION. ONE BAY PARTIALLY OPEN.

This project is one of the largest attempts at flood control engineerin dertaken in the United States. Its basic purpose is to prevent the adoptio the Mississippi River of the shorter route to the sea offered by the Atchaf River. The project includes a closure dike to permanently sever the na connection between the Mississippi and Atchafalaya Rivers via Old Riv gated control structure with inlet and outlet channels to provide for neces flow diversions from the Mississippi to the Atchafalaya under all condit and a navigation lock to preserve the navigational opportunities now affo by Old River.

The control structure consists of two independent units in order tha necessary flexibility in operation may be assured. The so-called low portion consists of eleven openings, 44 ft wide. Of these eleven open eight are 55 ft deep and three are 70 ft deep. All openings are controll multiple-leaf vertical lift gates, which will be manipulated to control low and medium stage flow. The overbank or high sill structure consisting bays, 44 ft wide and approximately 12 ft deep, equipped with needle type sures, will control diversion of high water and major flood flows. The structures are capable of passing a flow of approximately 700,000 cfs u existing channel conditions in the Mississippi and Atchafalaya Rivers. tinued development of the Atchafalaya River will increase this maximum pacity.

At the present time, the control structure, the outflow channel, and mo the levees are complete. The navigation lock is under construction, an entire project is scheduled for completion in 1964. However, should an e gency develop in which the need for closing Old River became immediate control structure could be operated.

In general, Old River Control will be operated in such a fashion as to tribute the flows artificially in almost the same proportions in which would have been distributed under the natural conditions that existed in The details of operation are not appropriate for presentation herein; how the entire operation will be oriented around the proposition of preservin existing stability of the Mississippi River channel below Old River. An o all aerial view of the salient project features is shown in Fig. 9.

OPERATION OF THE VARIOUS FLOODWAYS FOR DISTRIBUTING FLOOD FLOWS AT AND BELOW OLD RIVER

The natural elements of the flood control plan below Old River are operating. The artificial floodways and control structures are operat such a manner as to insure that the safe capacities of the various segmen the system are not exceeded. In the operation, guide lines are provide stages and discharges. There is only one specific operational criterion the sanction of law, and it is that the Bonnet Carre Spillway shall be ope to prevent stages at New Orleans from exceeding 20 ft.

The natural leveed channel of the Mississippi River at and below Orleans can accommodate all floods in which the peak flow in that reach river does not exceed 1,250,000 cfs. A flood producing this peak flow c expected to occur on an average of once every 5 to 7 yr, and would produ maximum stage at New Orleans of 20 ft. Such a flood represents the thre flood insofar as floodway operation is concerned.

r floods exceeding the magnitude of that previously described, operation : or more of the artificial floodways is required. In a typical flood the ıce of operations would generally be as follows.

Operation of either Bonnet Carre Spillway or Morganza Floodway when in excess of 1,250,000 cfs is in sight for the Mississippi River below iver.

Operation of the remaining Mississippi River floodway when a flow in s of approximately 1,500,000 cfs is in sight for the Mississippi River Old River.

Operation of the West Atchafalaya Floodway when the total flow at the le of Old River exceeds approximately 2,800,000 cfs.

OPERATIONAL PROBLEMS

he relationships between stage and discharge throughout the various seg- of the flood control system below Old River were constant, the opera- procedure would be straightforward and invariable. Whereas flow in

).—OLD RIVER CONTROL. LOW-SILL CONTROL STRUCTURE AND OUTLET CHANNEL IN CENTER OF VIEW. HIGH-SILL CONTROL STRUCTURE TO THE RIGHT.

l rivers follows the same immutable laws governing all fluid flow, al changes in the hydrography of these streams produce continuing s in their hydraulic efficiency. In addition to well-defined changes of a ssive nature, such as those taking place on the leveed portion of the alaya River, other changes for which cause-and-effect relationships ot been established combine with the aforementioned, and operate to n inflexible operational procedure impracticable. Notable among these variation in stage-discharge relationships for the Mississippi River it-

The intake capacities of all the artificial floodways are affected by changes previously mentioned. The Morganza Floodway intake capaci indicated to be in the neighborhood of 50,000 cfs per ft of stage for pr design conditions. Inasmuch as the stage-discharge relationship for the sissippi River below Old River has varied by as much as 2 ft for large fl the extent to which these variations influence diversions through Morg Floodway can be readily appreciated. The significance of these variatio enhanced when it is considered that they tend to vary from one section o river to another. For example, the stage discharge relationship in the sissippi River between Old River and Baton Rouge was, during the 1950 f as low as it had been for the entire period of record, whereas such was the case below Baton Rouge.

The performance of the West Atchafalaya Floodway is influenced by ations in stage-discharge relationships to an even greater extent. For e ple, as recently as 1941, the planned distribution of the project design anticipated simultaneous flows of 650,000 cfs and 250,000 cfs, in the Atchaf River and the West Atchafalaya Floodway, respectively. In 1950 the along the fuseplug levee at the head of the floodway averaged 5.5 ft belo fuseplug levee grade even though the peak flow in the Atchafalaya River re 650,000 cfs. The intake difficulty imposed by the steady lowering of the s discharge relationship for the Atchafalaya River at Simmesport, La. w worsened as time goes by.

The general problem of properly distributing floods below Old Rive volves providing answers to the following questions.

1. At what time during the flood should the first floodway be operated
2. Which floodway should be operated first?
3. At what time during the flood should the second floodway be oper
4. If the flood is indicated to be of extreme magnitude, what steps s be taken to insure that the West Atchafalaya Floodway operates in a ti and effective manner?

Time of Initial Operation.—This problem involves careful considerati antecedent stages and discharges on the Mississippi River. Because pr stages will be approached along the entire Mississippi below Bonnet C Spillway prior to the operation of the first floodway, whereas Atchaf Basin stages will be relatively moderate at that time, control and limit of stages in that reach or river are the primary concern. Accordingly operation must be closely coordinated with the stage at New Orleans. narily, a fairly definite decision can be reached as to the proper timing initial operation. Although unusual, cases will arise in which the proper of initial operation will be somewhat obscure. This could happen in a cresting at a discharge slightly larger than 1,250,000 cfs in the Missis River, particularly if the flood hydrograph rose rapidly. Such a flood probably produce unusually low stage-discharge relationships on the Mi sippi River in the early phases of the flood, in which case there would tendency to delay the operation in the hope of avoiding it altogether. This cedure could result in waiting too long, and thereby permitting the New Or stage to go above 20 ft. On the other hand, early operation based on charges alone, might result in an operation that could otherwise be avo

No all-inclusive specific rule can be laid down for time of initial oper but, in general, it would appear proper to base a Morganza first operati Mississippi River discharge, and a Bonne Carre first operation on New Or

Thus, if Morganza Floodway were to be operated first, this operation come sufficiently in advance of the Mississippi River discharge reaching 000 cfs to insure that not more than 1,200,000 cfs would flow past New ns. If Bonnet Carre Spillway were to be operated first, the operation come at a stage of approximately 19+ ft at New Orelans, with more than n prospect.

ich Floodway to Operate First.—This question is a great deal more com- han that of when to operate. There are advantages and disadvantages to perational sequences. The operation of Bonnet Carre Spillway is physi- quite simple. The land on which the spillway is located is owned in fee e by the federal government, and the problem of evacuation is nil. The t Carre operation involves little or no economic or commercial dis- n. Although unanimity of opinion does not exist with respect to the ef- f a spillway operation on fish and wildlife opportunities, most informed ts agree that operation of the spillway is helpful in this regard. Based perienced flows in Bonnet Carre Spillway and theoretical computations odel studies for Morganza Floodway, it is indicated that the magnitude flood that the overall system can safely accommodate with only Bonnet Spillway in operation is greater than that which would be accommodated nly Morganza Floodway. In practical terms, this means that floods ng near 1,500,000 cfs in the Mississippi below Old River could be handled Bonnet Carre alone without exceeding 20 ft at New Orleans, but not with nza alone.

e most serious disadvantage of a Bonnet Carre operation lies in the sed- ation concommitant to operation. In addition, the stage lowerings real- rom a Bonnet Carre operation are limited essentially to the Mississippi below the Spillway itself.

e Morganza Floodway, on the other hand, provides stage lowerings and ts in the Red River backwater area as well. Based on present best esti- the sedimentation problem in Morganza Floodway will be much less s than that in Bonnet Carre Spillway.

the unfavorable side, operation of the Morganza Floodway is much more ex than that of Bonnet Carre Spillway, and its effects will be felt over a r area. There is a substantial evacuation problem involved. Backwater g within the West Atchafalaya Floodway Area will be increased. Drain- the Pointe Coupee Protected Area, and in the towns of Melville and Springs, will be interrupted or impaired, and although dedicated areas ding runoff have been provided, there is always the outside chance that reme rainfall will overtax the storage provided.

an overall basis, it would appear that the advantage lies with a Bonnet first operation. However, it must be borne in mind that sedimentation ell be a controlling element. In any event, it is planned to operate the nza Floodway first in the next flood requiring operation of a floodway, r that the definite information needed to calibrate the control structure, ine overall hydraulic effects, and evaluate the sedimentation problem antitative manner may be collected.

e of Operation of Second Floodway.—The factors governing this prob- e similar to those relating to time of operation of the first, except that eshold discharge will be approximately 1,500,000 cfs. As before, if nza is to be operated, the operation would be keyed generally to the rge, whereas a Bonnet Carre operation would follow more closely the rleans stage.

Operation of West Atchafalaya Floodway.—Although the West Atchaf Floodway is not subject to control in the same sense that Morganza Floo and Bonnet Carre Spillway are, it nonetheless presents a most comple erational problem. The central element in this problem is the leveed ch of the Atchafalaya River. As mentioned previously, the operation of the Floodway is automatic and will result from the stage along its fuseplug exceeding the levee grade. The natural enlargement of the Atchafalaya has progressively lowered the stage along the fuseplug for a given total at the latitude of Old River. Although such a development may be view generally favorable, in that it is producing a continuous increase in the all intake capacity at the head of the Atchafalaya Basin, it does introdu impediment to timely operation of the West Atchafalaya Floodway, bec such operation must await the natural overtopping of the fuseplug. Failu the West Atchafalaya Floodway to operate on time would probably res overtaxing the safe leveed capacity of the Mississippi River, particularl low New Orleans.

At the present time, it appears that operation of the West Atchaf Floodway will commence early enough in a great flood to limit Missis River flows to a safe value, provided that the initial overtopping of the plug is followed promptly by various measures designed to facilitate ent floodwaters into the Floodway. Such measures might include degrading fuseplug levee at the head of the floodway or its destruction by explos Similar action may be taken along the West Atchafalaya River levee, i quired. However breaching the West Atchafalaya River Levee shou avoided, if possible, because it would produce excessive velocities i Atchafalaya River above the breach, and could result in uncontrolled caving and loss of the bridge across the Atchafalaya River at Simme

In view of the fact that continued improvement of the Atchafalaya Riv expected, the West Atchafalaya Floodway problem will tend to grow pro sively worse. The long-term solution lies in revision of the grades of l protecting the floodway.

CONCLUSIONS

Any plan for the control of floods below Old River must possess the fo ing attributes:

1. It must be capable of safely distributing the maximum peak flow may reasonably be expected to occur.
2. It must accomplish this distribution with a minimum of economi ruption.
3. It must be adaptable to changing conditions, so that its effectiv may remain unimpaired in the future.

The system described herein is shown to satisfy these requiremen can be operated in a planned and predictable manner for peak total fl the latitude of Old River up to and including 3,030,000 cfs, a value tha been established by extensive meteorological and hydrological studies largest likely to occur. With the exception of the West Atchafalaya Floo which will operate less often than once every hundred years, the economi ruption deriving from system operation is not great. It varies from negl in the case of an operation of Bonnet Carre Spillway to minor for a Mor

lway operation. And the overall system's flexibility and adaptability are that the end of its useful life cannot now be foreseen. With the system ting, the threat of damaging overflow below Old River has been virtually nated.

Journal of the
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Proceedings of the American Society of Civil Engineers

DICTING STORM RUNOFF ON SMALL EXPERIMENTAL WATERSHEDS

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SYNOPSIS

reful estimates of rates and amounts of runoff are required for design od retarding and channel stabilizing structures, and bridges and culverts all upstream watersheds. The period of record of many of the small ex- ental watersheds in the United States is too short to permit direct analy- r reliable estimates of the magnitude and frequency of storm runoff. A d is presented for extending the period of runoff records based on analy- existing short term records of rainfall and runoff for the watershed, and ger record of the rainfall alone. The method involves: (1) estimating runoff volumes from the rainfall pattern and antecedent rainfall, and stributing this runoff through an adaptation of the unit hydrograph princi- method is also presented for developing synthetic unit hydrographs for ed areas.

INTRODUCTION

ery limited amount of runoff data is available on small watersheds, and records are not generally of sufficient length to provide reliable esti- for design floods. Since records of precipitation are generally avail- or much longer periods than runoff records, it would be desirable to es- h precipitation-runoff relations and use such relations to develop flood

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ydr. Engr., Watershed Tech. Research Branch, Soil and Water Conservation Re- Div., Agric. Research Service, U. S. Dept. of Agric., Madison, Wis.

hydrographs from the longer precipitation record. The method presented is summarized as follows:

(1) Develop time-retention curves for each soil cover combination, b on an antecedent precipitation index and season.

(2) Estimate the storm runoff volume from the precipitation pattern an time-retention curve.

(3) Derive unit hydrographs for various rainfall intensities.

(4) Apply these relationships to a number of outstanding rainfalls and c pare the computed hydrographs with the observed runoff.

The relationships developed can be applied to a longer precipitation re from a nearby recording rain-gage. A relationship has also been develop permit adaptation of these principles to ungaged areas.

The analysis for developing the procedures was made with rainfall and off data from three small experimental watersheds located near Edwards Ill. (Fig. 1).

DESCRIPTION OF EXPERIMENTAL WATERSHEDS

The three Edwardsville watersheds, with a large number of runoff per each year, offer an excellent opportunity for studies of rainfall-runoff r tions on small watersheds. These watersheds (details shown in Fig. 2) designated as: W-1, a 27.2-acre cultivated area; W-2, a 50-acre past area; and W-4, a 290-acre mixed cover watershed. The two small waters are part of the 290-acre area. These watersheds are, in general, quite in the upper portions but drop off rather abruptly near the waterways.

Watershed W-1 is a fan-shaped area with a range in elevation of 20 ft, more than two-thirds of the area has slopes less than 1%. This area was cultivated during the years of 1938 and 1948-50, 80% cultivated in 1951 1955, 100% alfalfa in 1940 to 1943, and 44% cultivated during the rest o period of record. Areas not cultivated were in legume hay.

Watershed W-2 is an oval-shaped area having a length about one and half times its width. The difference in elevation between the divide an runoff station is 39 ft. About one-third of the area has an average slo 1.5%, and another third has 12% slopes. During the period of record, cove this area has varied from 0 to 15% cultivated with the remainder in pastu

Watershed W-4 is nearly circular and has a range in elevation of 63 ft. third has an average slope of 1%, and another third has 12% slopes. The m mum slope near the waterway is 35%. There are four small ponds with a drainage area of approximately 30 acres or 10% of the watershed area. ing the period of record, from 17 to 30% of this area has been cultivate the remainder mostly in pasture.

The soils on these watersheds are of loessial origin over glacial till an impervious claypan layer at depths of 10 to 20 in. These soils, whe and protected with a good vegetative cover, will take in precipitation r rapidly until the surface layer becomes saturated, after which additional cipitation will result in a high percentage of runoff. The data obtained on areas are applicable to the central claypan areas of southern Illinois, sou Indiana, southwestern Ohio, Missouri, eastern Kansas, and northeastern homa. The mean annual precipitation over most of this area is about 4 but the mean monthly rainfall varies widely from a fairly uniform distrib

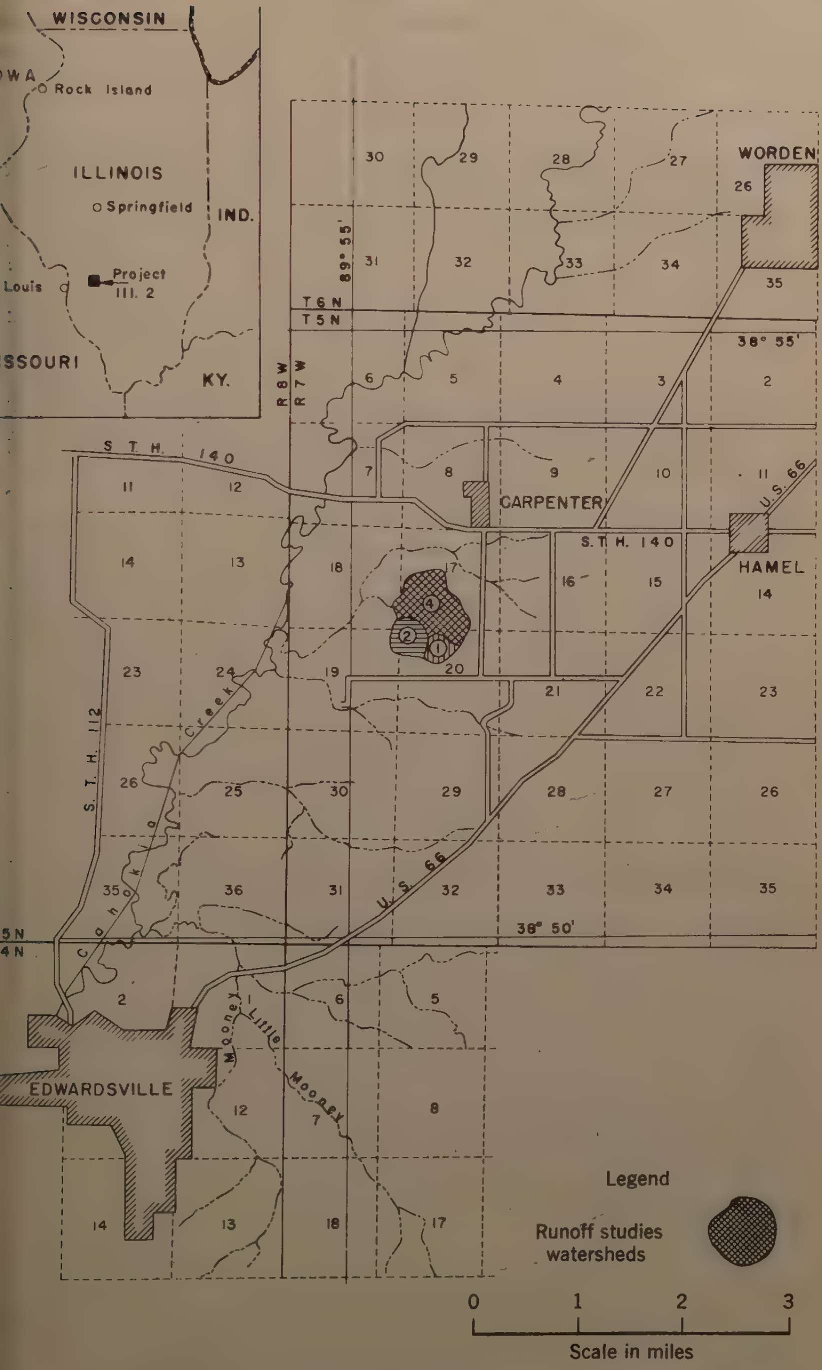

FIG. 1.—LOCATION OF WATERSHEDS

in southern Illinois, to eastern Kansas, which has high spring and summe low winter precipitation.

BASIC DATA

Continuous records of precipitation, runoff, temperature, humidity, and cover were collected on the three Edwardsville watersheds from March, 1 through December, 1955.

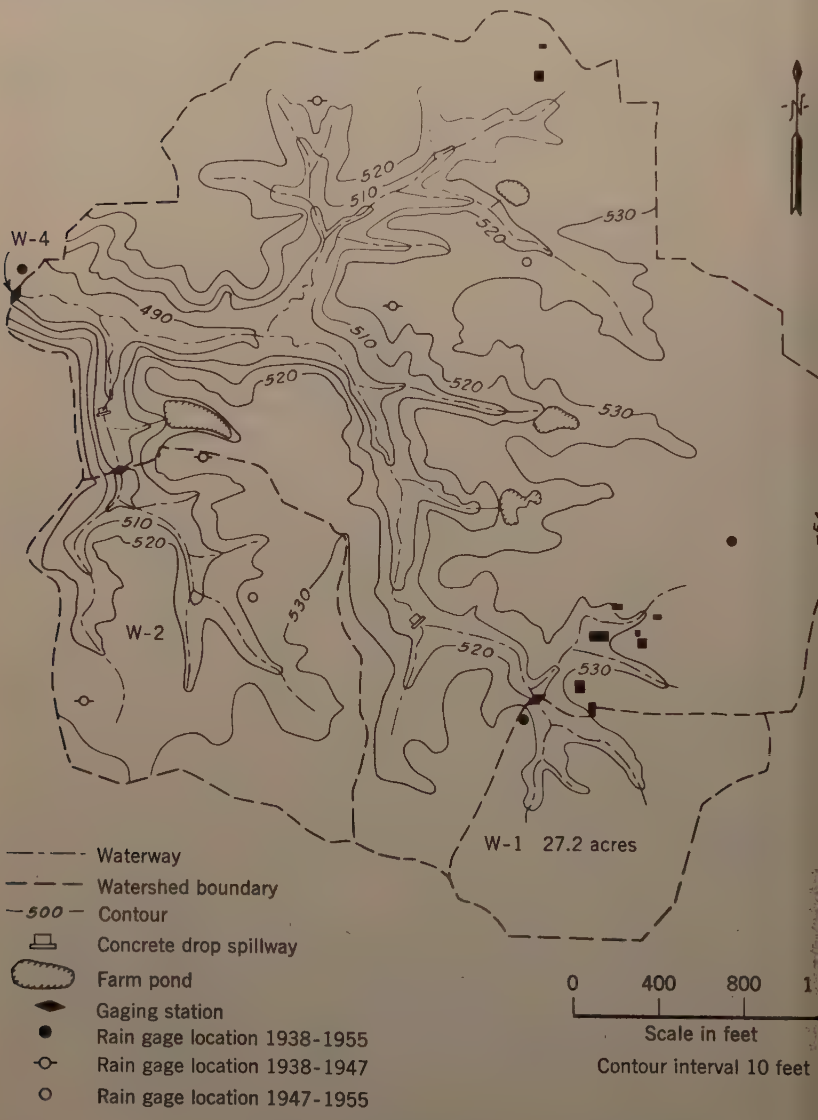

FIG. 2.—MAP OF EDWARDSVILLE, ILL., WATERSHEDS

recipitation was measured with seven recording raingages from 1938 to and five recording gages from 1948 through 1955. Additional recording s were in operation on the two small areas during an intensive infiltration from 1940 to 1943. The only gage on the 27-acre area, except for 1940-vas located near the runoff station. Because of the variation in rainfall ng thundershowers, this gage may not be representative of the actual aver-precipitation on the area.

unoff from these areas was measured with concrete V-notch broad crest-eirs equipped with water stage recorders. Runoff records from watershed are based on the effective area varying in size from 290 acres, when all s are at spillway elevation prior to rainfall, to 260 acres when none of the s are discharging through the spillway.

efinitions.—The terms used herein are defined as follows:

Retention - That part of the precipitation not appearing as runoff, in-es plant interception, depression storage, and infiltration.

Antecedent Precipitation Index (API) - An empirical measure of the ef-of precipitation falling a given number of days prior to the period of ex-

Runoff - As used here includes all flow that eventually passes over the including surface runoff, interflow, and base flow.

Unit Hydrograph - A hydrograph representing 1 in. of runoff.

Time to Peak - The time from beginning of excess rainfall to the peak of runoff for storms of short duration.

Period of Excess - The time during which precipitation occurred at rates cess of the infiltration capacity.

Infiltration Capacity - The maximum rate at which a given soil, when in en condition, can absorb falling rain.[2]

ESTIMATING STORM RUNOFF VOLUMES

alysis was first directed toward developing a relationship between ante-t precipitation (as a measure of the available soil moisture storage) and tion.

mputing Antecedent Precipitation Index.—The length of time that must be dered in computing any antecedent precipitation condition will vary with ermeability of the soils and subsoils, temperature, evaporation, transpi-, and vegetation. For very permeable soils such as sands or the better oams, a 5-day antecedent precipitation index may suffice, whereas, for a y permeable soil it may be necessary to use a period of 30 days or longer. us methods and lengths of period were tested in computing the antecedent pitation index for Edwardsville areas, and the one that gave the best corre-with retention was a 30-day index plus 20% of the departure from normal 30-60 day precipitation. The final selection which gave nearly as good on with less computation was based on a 30-day period using the formula:

$$API = P_o + P_1k + P_2k^2 + P_3k^3 + - - - - P_nk^n \quad \ldots\ldots\ldots \quad (1)$$

ch API is the antecedent precipitation index, P_o refers to precipitation 24 hr prior to the excessive period which produced the runoff shown in

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Surface Runoff Phenomena, Part I—Analysis of the Hydrograph," by Robert E. Hor-ublication 101, Horton Hydrological Lab., Voorheesville, N. Y.

column 6 at Table 1, P_1, P_2, and P_3 indicate precipitation 1, 2, 3 days, and on, prior to the storm, n denotes the number of days used to establish the dex (in this case 30), and k is a constant depending primarily on the soil t (0.95 gave the best results for the claypan areas).

Relation of Antecedent Precipitation to Retention.—Initial investigati showed that a large volume of data was needed to develop relationships betw antecedent precipitation index and retention. Therefore, consideration was gi to combining the data from watersheds W-1, W-2, and W-4. Compariso amounts of precipitation retained from the excessive portion of storms show that retention on W-1 (27.2 acres - all alfalfa), during 1940-1943 was ab 95% of that on W-4. When W-1 was 96% cultivated, the retention for individ storms averaged 18% less than W-4. Retention on W-2 (50 acres - of all p ture) averaged 5% more than on W-4. Comparison of annual and seasonal r offs on watersheds W-1 and W-2 against W-4 show that for the period of 19 1955, the average runoff on the two small areas was only 9% more than on 4. With the apparent small variation between watersheds, it was felt that d from all three areas could safely be combined for the retention analysis.

Table 1 gives the data for watershed W-4 for all storms during the 18 y record that caused runoff in excess of 0.1 in. and where the precipitation such that all runoff could be charged to a definite period of rainfall exce Where it was possible to break a storm down into several distinct runoff pe ods, each of these periods was entered in the table. Some winter storms w eliminated because of frozen ground or melting snow. Similar informatio that presented in Table 1 was compiled for watersheds W-1 and W-2.

The data for all watersheds were divided into several groups accordin season and further subdivided by groupings based on the API. In grouping th data according to season, one should recognize that variation in season fr one year to the next may be as much as two weeks. Because of this, it is entirely logical to arbitrarily select a given date as the limit of a particu group. The year was first divided into spring, summer, fall, and winter three subdivisions for API in each, giving a total of 12 groups of data.

The data from columns 4 and 6 of Table 1 and similar information from W and W-2, grouped as explained previously, were plotted on rectangular coor nate paper. The results indicated a rapidly changing infiltration capacity d ing the spring and fall seasons and fairly constant conditions during the su mer and the dormant season. They also indicated the advisability of sligh different grouping according to seasons. The final groupings, selected as g ing good results with a minimum of groups, were: November 15 to April April 15 to May 15; May 15 to June 30; July through September; and October to November 15.

Fig. 3(a) shows these time retention curves for the period July through S tember at different antecedent precipitation indexes. Curves were fitted i the groups of points as plotted. Similar sets of curves were prepared for e of the periods listed. Fig. 3(b) shows the maximum retentions that result fr antecedent precipitation indexes less than 0.5. Fig. 3(c) shows retentions the various periods, termed average because they occur more frequently t the others. To continue these curves to the origin of time and depth wo serve no purpose. Those representing dry or summer conditions would br down very sharply, indicating the magnitude of plant interception, depress storage and initial infiltration rate. Minimum infiltration rates on these s are nearly constant throughout the year varying between .02 and .06 in. per

TABLE 1.—DATA FROM SELECTED STORMS CAUSING MORE THAN 0.1 IN. RUNOFF, 1938-55

Date	Precip. before excess Inches	Excessive Precipitation			Runoff from excess, Inches	Retention from excess, Inches	30-day Antecedent Precipitation Index (A.P.I.)
		Amount, Inches	Duration Hrs.	Duration Min.			
(1)	(2)	(3)	(4)		(5)	(6)	(7)
13, 1938	.04	.85	5	30	.17	.68	1.78
13,	0	.76	2	40	.44	.32	2.59
15,	0	.59	2	07	.28	.31	2.87
15,	0	.45	0	28	.39	.06	3.43
22,	0	.66	0	22	.20	.46	2.46
28,	0	.58	3	50	.18	.40	2.54
8,	.10	.68	1	45	.52	.16	4.95
27,	.04	1.30	0	32	.63	.67	2.68
10,	0	.71	0	08	.20	.51	2.77
23,	0	.68	0	55	.16	.52	2.36
17,	.93	3.28	2	36	1.33	1.95	1.48
9, 1939	.53	.36	0	13	.27	.09	2.03
19,	.03	.93	5	30	.37	.56	1.55
11,	.15	.61	2	17	.18	.43	1.26
1,	0	.40	1	50	.27	.13	1.86
5,	0	.55	4	00	.33	.22	1.99
20,	0	.43	5	00	.13	.30	2.45
6,	.55	1.98	1	20	.58	1.40	2.46
1,	.08	.84	0	12	.22	.62	2.64
1,	0	.58	0	40	.19	.39	3.39
2,	.25	.47	0	45	.16	.31	4.38
7,	.07	4.42	9	20	1.80	2.62	3.25
0, 1940	.05	1.14	4	07	.35	.79	1.49
4,	.05	3.58	2	35	.73	2.85	.19
7, 1941	.53	.49	0	13	.16	.33	2.23
0,	0	1.25	0	55	.25	1.00	2.21
2,	0	1.43	1	00	.21	1.22	1.58
2,	0	.57	0	25	.24	.33	2.88
9,	.07	.64	0	17	.10	.54	2.22
2,	.04	.49	0	20	.12	.37	3.45
2,	.59	.54	0	40	.29	.25	3.98
5,	0	1.95	17	00	.78	1.17	3.53
5, 1942	.10	1.00	1	45	.19	.81	1.58
3,	.05	1.16	0	33	.40	.76	1.52
1,	0	1.39	0	25	.69	.70	2.36
6,	.05	1.05	0	20	.71	.34	3.88
6,	.05	1.70	4	15	.99	.71	3.88
8,	.05	2.50	1	20	1.78	.72	3.14
8,	.05	3.85	7	25	2.37	1.48	3.14
8,	2.50	.75	0	30	.52	.23	5.64
9,	0	.22	1	50	.09	.13	7.33
9,	0	.81	1	30	.57	.24	7.32
4,	0	.56	2	25	.12	.44	4.58
6,	0	1.06	0	23	.33	.73	2.52
7,	0	.77	0	25	.31	.46	3.62
7,	.07	.69	0	18	.25	.44	2.21
7,	.20	2.31	6	50	1.64	.67	.60
3, 1943	0	.60	2	30	.17	.43	.24
5,	0	.48	3	10	.23	.25	1.08

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)		(5)	(6)	(7)
Mar. 16,	.48	.34	1	20	.24	.10	1.54
Mar. 19,	0	.52	3	30	.36	.16	1.89
May 6,	.95	1.00	2	10	.39	.61	1.93
May 8,	0	.30	1	30	.12	.18	2.80
May 10,	0	.41	1	35	.36	.05	3.61
May 15,	.35	.35	0	12	.15	.20	3.90
May 16,	0	.29	0	07	.16	.13	4.03
May 17,	0	5.05	29	00	3.96	1.09	4.30
May 17,	.50	4.55	24	00	3.55	1.00	4.75
May 17,	.50	1.85	6	40	1.44	.41	4.75
May 17,	2.65	1.00	1	45	.81	.19	6.50
May 17,	3.70	1.65	7	40	1.29	.36	7.50
May 19,	0	.60	8	15	.47	.13	8.1
June 5,	0	.87	2	45	.12	.75	4.46
June 6,	0	1.43	3	30	.82	.61	5.04
June 10,	0	.73	0	50	.34	.39	4.8
Apr. 10, 1944	.06	.53	1	05	.08	.45	1.1
Apr. 22,	.22	4.67	14	00	3.15	1.51	2.15
Apr. 22,	0	1.01	4	00	.68	.33	6.90
May 22,	0	1.06	2	10	.19	.87	2.4
May 23,	0	.59	1	10	.13	.46	2.3
May 23,	0	.41	1	40	.13	.28	2.9
Aug. 30,	0	1.70	2	00	.22	1.48	1.9
Feb. 21, 1945	.13	.85	4	45	.18	.67	.8
Mar. 1,	0	1.49	16	20	.59	.90	1.8
Mar. 6,	0	.64	4	00	.38	.26	3.9
Mar. 25,	0	1.78	18	30	.75	1.03	2.1
Mar. 29,	0	1.32	1	55	.61	.71	2.9
Mar. 30,	0	1.43	21	00	.96	.47	4.3
Apr. 1,	0	.54	6	15	.34	.20	5.1
Apr. 12,	.10	1.31	5	00	.42	.89	3.1
June 9,	0	.52	0	13	.27	.25	3.1
June 12,	0	.87	1	30	.33	.54	3.2
June 16,	0	2.20	1	50	1.33	.87	3.1
June 16,	0	.32	0	18	.18	.14	5.4
June 17,	0	.24	2	00	.14	.10	5.6
June 17,	0	.34	4	40	.20	.14	5.8
Oct. 21,	0	1.44	6	30	.22	1.22	1.0
Nov. 12,	.05	.64	0	40	.16	.48	1.0
Nov. 26,	0	.57	1	35	.12	.45	1.1
Dec. 24,	0	1.10	17	30	.33	.77	.5
Jan. 11, 1946	0	.53	4	45	.24	.29	1.3
Feb. 12,	0	1.72	21	00	.87	.85	.4
Mar. 5,	.10	1.40	5	00	.74	.66	1.1
Mar. 17,	0	1.37	14	00	.65	.72	1.2
May 3,	0	.64	0	30	.27	.37	2.2
May 17,	0	1.05	6	05	.35	.70	2.7
Aug. 5,	0	1.12	0	38	.17	.95	2.4
Aug. 14,	.15	1.49	3	40	.32	1.17	4.0
Aug. 14,	.20	3.09	2	20	2.31	.78	5.6
Aug. 14,	.20	5.00	6	05	3.41	1.59	5.6
Aug. 14,	.20	1.10	1	15	.67	.43	5.6
Aug. 15,	0	1.45	0	50	1.10	.35	9.2
Aug. 15,	.30	3.66	10	40	2.54	1.12	10.5
Oct. 31,	.35	1.49	6	30	.14	1.35	1.1
Oct. 31,	0	4.01	15	45	1.75	2.26	2.6
Nov. 2,	0	.91	4	20	.50	.41	6.3

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)		(5)	(6)	(7)
6,	0	1.05	9	30	.44	.61	6.40
9,	0	1.45	8	00	.89	.56	6.40
15,	.06	.50	5	00	.15	.35	5.59
12,	0	1.00	6	40	.70	.30	1.31
29, 1947	0	.93	3	25	.47	.46	.32
3,	0	.48	4	30	.25	.23	1.22
5,	0	.53	3	30	.39	.14	1.57
10,	0	.81	7	05	.33	.48	1.70
24,	0	1.59	15	00	.98	.61	1.77
29,	0	1.37	1	43	.95	.42	3.16
1, 1948	0	.55	3	55	.10	.45	.77
21,	0	.91	6	00	.34	.57	.96
22,	0	1.13	11	20	.59	.54	1.78
26,	0	.57	1	35	.30	.27	2.39
30,	0	.70	6	50	.30	.40	2.68
31,	0	.26	2	00	.17	.09	3.20
15,	0	.76	1	20	.18	.58	2.04
26,	0	.84	0	50	.12	.72	1.76
27,	0	.59	0	30	.13	.46	2.63
15,	0	.54	0	10	.10	.44	3.07
19,	0	.52	0	30	.14	.38	3.55
20,	0	.85	1	10	.35	.50	4.23
25,	0	2.45	6	10	1.10	1.35	4.59
25,	0	1.85	1	45	.85	1.00	4.59
3, 1949	0	.98	4	05	.34	.64	.02
18,	0	1.81	12	00	1.01	.80	1.34
9,	0	.81	0	15	.13	.68	.98
9,	0	1.10	0	23	.58	.52	1.78
21,	0	2.02	2	10	.68	1.34	2.32
26,	.15	1.35	1	05	.27	1.08	1.18
27,	.04	1.00	0	30	.41	.59	2.30
2,	0	1.86	1	10	.49	1.37	1.47
5,	.45	1.67	4	00	.22	1.45	1.83
11,	1.10	.85	5	00	.22	63	3.85
11,	0	.42	0	20	.10	.32	2.89
25,	0	.59	4	15	.25	.34	2.70
3, 1950	0	4.66	25	00	3.65	1.01	2.23
14,	0	.59	5	30	.46	.13	5.19
25,	0	.55	2	40	.13	.42	2.89
11,	.25	1.07	5	10	.60	.47	1.30
12,	0	.54	5	00	.33	.21	2.33
19,	0	.52	6	40	.32	.20	2.31
28,	.10	.74	2	50	.19	.55	1.54
29,	0	.86	1	20	.24	.62	1.57
3,	0	1.50	6	30	.62	.88	2.35
3,	.50	.98	3	00	.51	.47	2.85
13,	0	1.27	1	30	.33	.94	2.49
14,	0	.62	1	40	.26	.36	3.58
18, 1951	0	.52	3	00	.19	.33	1.72
20,	0	1.65	15	30	.70	.95	2.12
16,	0	1.74	2	00	.62	1.12	2.42
19,	0	.85	1	45	.14	.71	3.88
22,	.10	.50	0	25	.11	.39	4.55
23,	0	.92	0	13	.21	.71	4.42
28,	0	.92	0	35	.43	.49	4.46

TABLE 1 (CONT'D)

(1)	(2)	(3)	(4)		(5)	(6)	(7)
July 22,	0	1.04	0	20	.18	.86	1.33
July 23,	0	.90	0	15	.38	.52	2.20
July 23,	0	.69	0	15	.42	.27	3.15
Sept. 12,	0	.90	0	35	.12	.78	1.29
Jan. 26, 1952	.10	.34	0	15	.11	.23	.39
Feb. 1,	0	.75	5	00	.27	.48	.53
Mar. 31,	.22	1.36	0	35	.97	.41	1.50
Mar. 31,	.22	2.73	2	47	2.01	.72	1.50
Mar. 31,	1.65	1.25	0	30	1.04	.21	3.15
Apr. 3,	.07	1.28	13	30	1.00	.28	3.49
Apr. 12,	0	1.55	17	00	1.30	.25	3.20
June 9,	0	2.27	1	15	.80	1.47	.30
June 9,	0	3.11	2	15	1.24	1.87	.30
June 9,	0	3.60	3	15	1.45	2.15	.30
June 9,	2.30	.78	0	15	.42	.36	2.90
June 9,	3.12	.50	0	30	.22	.28	3.42
July 2,	.15	2.92	1	10	1.11	1.81	1.62
July 2,	.75	2.34	0	30	1.11	1.23	2.22
July 2,	0	.76	0	23	.27	.49	4.54
July 16,	0	1.55	2	17	.40	1.15	3.31
Mar. 3, 1953	.07	1.05	7	00	.40	.65	1.15
Mar. 17,	0	.55	4	30	.11	.44	1.35
Mar 30,	0	.78	5	00	.11	.67	1.05
Mar. 31,	0	.36	0	35	.12	.25	1.73
Apr. 23,	.27	1.20	1	05	.60	.60	1.49
June 26,	0	2.78	2	40	.58	2.20	.26
June 26,	0	1.58	0	45	.14	1.44	.26
June 26,	1.58	1.20	1	00	.44	.76	1.84
Oct. 5, 1954	0	.85	0	20	.16	.69	1.45
Oct. 11,	.85	.50	0	30	.11	.39	2.70
Oct. 11,	.10	1.55	5	00	.56	.99	3.25
Oct. 26,	.33	.62	0	45	.20	.42	2.73
Feb. 19, 1955	.40	.42	2	15	.22	.20	.64
Feb. 19,	0	.50	3	30	.18	.32	1.00
Feb. 26,	.05	.55	2	30	.11	.44	1.30
Apr. 23,	.05	.70	0	21	.15	.55	.97
July 17,	.05	1.55	0	48	.29	1.26	2.15

These studies show that differences in antecedent precipitation have co siderable influence on retention during the summer but are of minor importa in the winter. The best correlation between antecedent precipitation index a retention was obtained for the period of October 15 to November 15, but t might change markedly if a larger number of points were available. The curves are based on the assumption that, for a particular season, the infilt tion capacity is a function of antecedent soil moisture and is independent rainfall intensity. Because these are mass retention curves, the slope of tangent is a measure of the infiltration capacity rate.

Because it is the amount of precipitation retained rather than the total pr cipitation that adds to the soil moisture, a refinement was attempted using antecedent retention index. The results showed no noticeable improvement

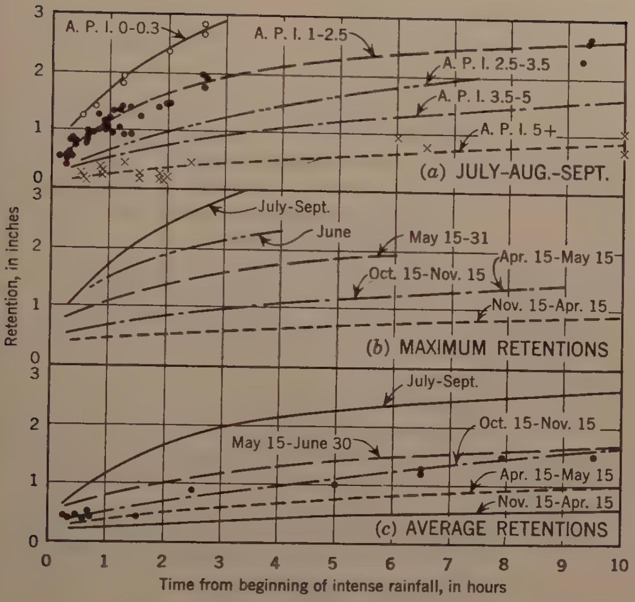


FIG. 3.—TIME-RETENTION CURVES (CLAYPEN PRAIRIES)

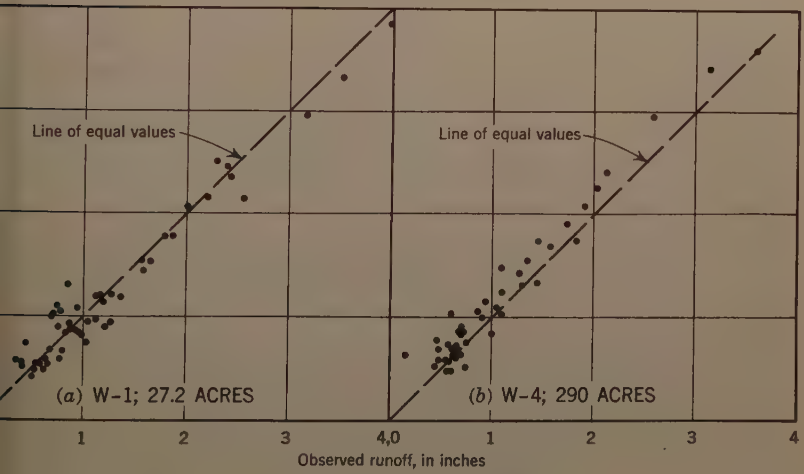


FIG. 4.—COMPARISON OF ESTIMATED AND OBSERVED RUNOFF

The investigations also indicate that data from a 187-acre watershed Hamilton, Ohio, and the 153-acre area on the McCredie, Mo., experiment tion agree quite favorably with the retention curves developed fromthe Edwar ville data. The soils on these areas are of the same general type as at Edwa ville, but depths to the impervious claypan may be somewhat different.

Testing the Retention Curves.—To test the accuracy of the retention cur the estimated runoff volumes for a number of storms were obtained by pla the time retention curve for the appropriate season and antecedent preci tion index under the mass rainfall diagram, with the zero time and depth a beginning of rainfall excess. In the case of watershed W-4, the mass rai diagram was the average of all gages. The vertical distance between the retention and mass rainfall curves represents the estimated amount of rai excess, up to any time, which will eventually pass over the weir. Prope lowance must be made for the decrease in depression storage and plant in ception, when, during any periods within the storm, the rainfall ceases or below infiltration capacity.

A large number of selected storms that gave an estimated runoff of 1/ or more were tabulated and compared with the observed runoff. These c parisons for watersheds W-1 and W-4 are shown in Figs. 4(a) and 4(b), spectively. In general, the best agreement was obtained for storms occur during the dormant season and at other times of high antecedent moisture ditions, such as July 9, 1942, May 17, 1943, and August 15, 1946. Alth individual values show considerable variation between the estimated and a runoff, the totals for all storms on W-1 agree very well. The estimated amo on W-4 appear to be generally too high, indicating the possible need for adj ment of the time retention curves on this area.

DEVELOPING HYDROGRAPHS

After establishing a method for estimating storm runoff volume, the step in the analysis was to develop a method of estimating the shape of the drograph.

Rainfall Intensity and the Unit Hydrograph.—Attempts to establish an a age unit hydrograph for each of the Edwardsville watersheds showed a variation between storms having different rainfall intensities. It was desir therefore, to establish relationships between rainfall intensity and shape o unit hydrograph.

For the 27-acre area, a series of storms was selected having isolated p ods of intense rainfall of 10 to 15 min duration, of reasonably uniform inten and producing at least 0.10 in. of runoff. The data for these storms are s in Table 2.

The unit graph peak and time from beginning of rainfall excess to peak of runoff, t_p, were plotted against rainfall intensity as shown on Fig. 5. plotted data indicate two separate groupings, depending on whether the p of high intensity came at the beginning or late in the storm period. The on this figure (fitted by inspection) show that both the peak of the unit g and the time to peak are dependent on rainfall intensity and storm patter

To determine if similar relations existed on the larger areas, a seri storms was selected, for the 290-acre watershed, which had the foll characteristics: total storm runoff not less than 0.10 in.; reasonably uni rainfall intensity for the period of excess; and a duration of rainfall exces

Date	Antecedent Precipitation Index	Precipitation Before Period of Excess: Amount Inches	Precipitation Before Period of Excess: Duration Minutes	Precipitation Producing Unit Hydrograph: Time Minutes	Precipitation Producing Unit Hydrograph: Amount Inches	Precipitation Producing Unit Hydrograph: Intensity In/Hr	Runoff Used in Computing Unit Hydro.: Peak Rate In/Hr	Runoff Used in Computing Unit Hydro.: Amount Inches	Unit Graph Peak In/Hr	Time from Beginning of Excess Rain to Peak Rate of Runoff Minutes
May 27, 1938	2.93	.28	20	14	1.11	4.75	2.38	.66	3.60	12
June 10,	2.77	0	0	6	.60	6.00	.61	.20	3.05	14
Aug. 11, 1939	2.64	.06	88	10	.74	4.44	.48	.22	2.18	14
April 17, 1941	2.23	.58	132	13	.42	1.95	.25	.14	1.78	20
Sept. 2,	2.55	1.02	43	12	.53	2.65	.38	.17	2.23	18
Oct. 22,	4.52	1.02	295	10	.22	1.32	.14	.10	1.40	24
June 13, 1942	1.55	.24	33	24	.92	2.30	.20	.14	1.43	33
July 8,	5.64	2.48	140	15	.60	2.40	.94	.46	2.04	19
July 9,	6.89	0	0	26	.67	1.55	.60	.63	.95	28
Aug. 6,	2.35	.03	2	18	1.02	3.40	.34	.21	1.62	27
Aug. 7,	2.49	.04	2	10	.60	3.60	.33	.21	1.57	26
June 9, 1945	2.93	0	0	8	.44	3.30	.27	.20	1.35	22
June 16,	3.11	.10	7	17	.73	2.58	.90	.60	1.50	21
June 16,	5.37	2.26	210	10	.13	.78	.12	.12	1.00	22
May 3, 1946	2.24	0	0	13	.70	3.23	.54	.32	1.69	21
Aug. 3,	1.91	.81	255	10	.66	3.96	.26	.15	1.73	24
Aug. 5,	2.41	0	0	17	.81	2.86	.26	.17	1.53	29
July 15, 1948	3.07	.09	15	10	.53	3.18	.27	.12	2.25	24
July 20,	4.44	0	0	10	.52	3.12	.53	.34	1.56	23
July 20,	5.05	.63	52	17	.27	.95	.25	.21	1.19	30
May 9, 1949	.98	0	0	10	.80	4.80	.22	.08	2.75	15
May 9,	1.80	0	0	18	1.26	4.20	1.40	.75	1.87	20
June 26,	1.18	.37	62	13	.73	3.37	.44	.22	2.00	27
June 23, 1951	4.55	0	0	12	.75	3.75	.98	.40	2.45	
July 22,	1.29	0	0	19	1.15	3.63	.66	.40	1.65	34
July 23,	2.20	0	0	15	.95	3.80	.92	.59	1.56	23
July 23,	3.15	0	0	15	.75	3.00	1.07	.61	1.75	25
Mar. 31, 1952	1.52	.19	80	35	1.36	2.33	1.51	1.24	1.22	39
Mar. 31,	3.00	1.75	207	23	1.18	3.08	2.02	1.08	1.87	25

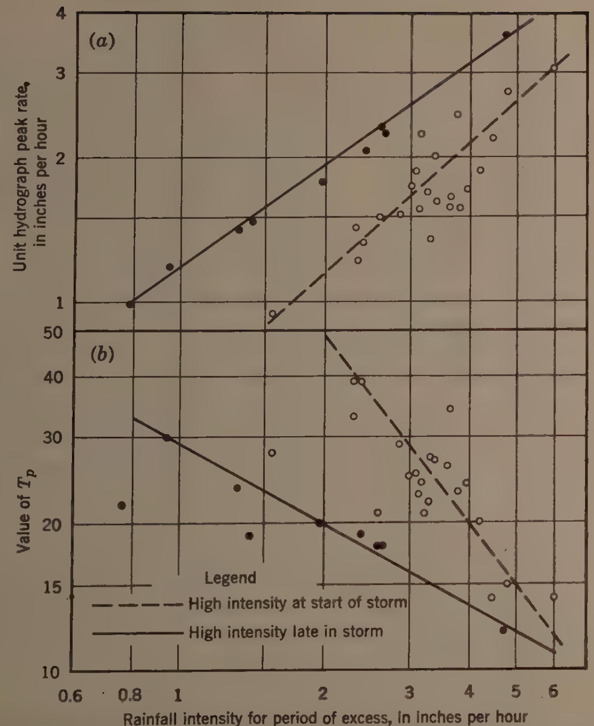


FIG. 5.—EFFECT OF RAINFALL INTENSITY AND

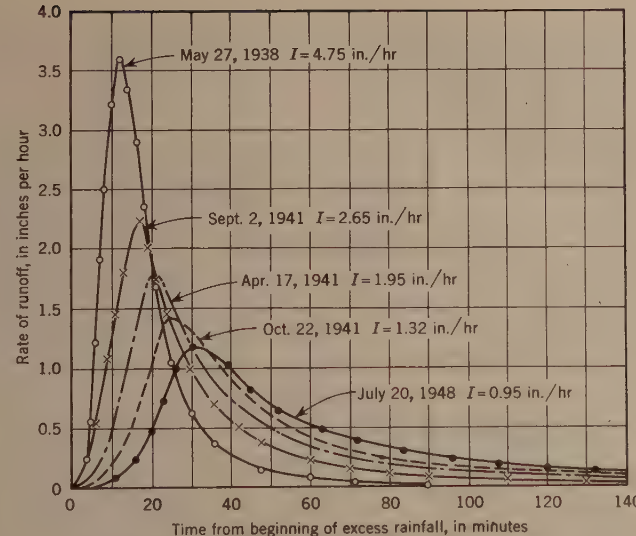
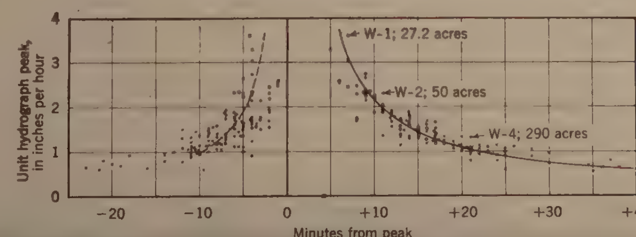


FIG. 6.—UNIT HYDROGRAPHS FOR WATERSHED W-1



than 10 min, or more than the time to peak. The group of storms select-
ad a duration of rainfall excess from 11 to 30 min with the exception of
54-min period on April 22, 1944.

rouping of the data according to the duration of the rainfall producing the graph, and plotting similar to Fig. 5, indicated that duration has little ing on the results so long as it is less than the time to peak. The plotted s again showed two separate groupings depending on whether the period gh intensity occurred early or late in the storm period. A comparison of lottings for these two watersheds showed that, for the larger watershed, all intensity has less effect on the peak of the unit graph and the time to

One might logically expect that the runoff hydrograph peak resulting rainfall of a given duration would be more closely related to the net rain- ntensity, that is, rainfall intensity minus rate of infiltration. Attempts to elate this net rainfall intensity with the peak of the unit graph and time to gave no improvement. This was probably due to the difficulty of making onable estimates of the infiltration rates for such short periods.

rom the graph of Fig. 5, a series of storms was selected to represent vari- ainfall intensities. Unit hydrographs for each of these storms are shown g. 6. The actual total runoff, for the storms selected, varied from 0.10 to in., with the exception of the storm of May 27, 1938, which had a total f of 0.66 in. The original unit hydrograph definition by L. K. Sherman,[3] ded for use on very large drainage areas states:

'or the same drainage area, however, there is a definite total flood riod corresponding to a given rainfall and all one-day rainfalls, re- rdless of intensity, will give the same length of base of the hydro- aph."

data presented here, which applies to very small areas, the base of the raph increases as the rainfall intensity decreases. Perhaps this is be- , for rains of less than 15 min., at low intensities, the soil must be at a r moisture content to produce a minimum of 0.10 in. of runoff than for of higher intensities. Only a small percentage of the runoff in these s was due to base flow or interflow since the rate 3 hr after the peak was bout 1% of the peak and, therefore, no attempt was made to separate the graphs into their component parts.

veloping Synthetic Unit Hydrographs.—With a limited amount of data ble and with considerable scatter of points, as shown in Fig. 5, one may ently find it impossible to pick a storm representative of a desired inten- r storm pattern.

tudy was made of the unit graphs to show times before and after the peak various percentages of the peak rate occurred. The data for watershed re shown in Table 3. Similar data from the other two Edwardsville water- were included in this analysis. The study showed two distinct groupings he times were compared with rainfall intensity. However, all points fell group when plotted against the unit hydrograph peak. A plotting of the of occurrence of rates equal to 60% of the peak is shown in Fig. 7, in - indicates minutes before peak and + indicates minutes after peak. The s show that, when times are plotted against the unit graph peak, one can adequately define the shape of the recession for all three watersheds.

treamflow from Rainfall by Unit-Graph Method," by L. K. Sherman, Engineering Record, Vol. 108, 1932, pp. 501-505.

TABLE 3.—TIMES FROM PEAK OF HYDROGRAPHS TO POINTS WHERE DISCHARGE EQUALS A GIVEN PERCENTAGE OF THE PEAK WATERSHED

Date	Rainfall		Unit Graph	Time in Minutes from Peak to--									
	Time Min	Intens. In/Hr	Peak In/Hr	80% Before	Peak After	60% Before	Peak After	40% Before	Peak After	25% Before	Peak After	15% Before	Peak After
Mar. 15, 1938	26	1.20	1.36	3	8	5	15	7	27	9	43	11	67
May 27,	14	4.75	3.60	3	5	4.3	7	5.3	10	6	15	7	20
May 28,	12	.55	2.34	4	6	5.5	9.5	7	15	8	30	9	46
June 10,	6	6.00	3.05	3	4	4	7	6	12	8	21	8	31
Aug. 11, 1939	10	4.44	2.18	1	5	1.5	10	2	21	2	32	3	48
Apr. 17, 1941	13	1.95	1.78	3	6	6-	12	8	20	10	33	12	47
Sept. 2,	12	2.65	2.23	4	5	5	9	7.5	16	10	24	11	35
Oct. 22,	15	1.68	1.89	5	7	7	11	9	18	11	27	12	38
Oct. 22,	25	.91	1.27	8	8	12	16	15	25	18	40	21	58
Oct. 22,	10	1.32	1.40	4	10	7-	17	9	28	12	39e	13	60
June 13, 1942	24	2.30	1.43	6	11	9	17	13	27	17	38	22	52
July 8,	15	2.40	2.04	3	6	4.5	11	5.5	20	7	33	9	57
Aug. 6,	18	3.40	1.62	4	8	6	14	9	24	12	37	15	58
Aug. 7,	10	3.60	1.57	5	9	8-	17	9	29	11	52	13	69
June 16, 1945	17	2.58	1.50	2	8	3	13	4	21	6	39	7	60
June 16,	12	1.50	1.19	5	8	7-	17	8	30	9	50	11	79
May 3, 1946	13	3.23	1.69	3	4	4	8	5	15	7	32	9	53
Aug. 3,	10	3.96	1.73	1	6	3	12	7	21	11	34	13	51
Aug. 5,	17	2.86	1.53	4	8	6	16	8	32	11	44	14	60
Aug 5,	20	1.65	1.54	3	7	4	14	5	24	6	40	9	63
June 26, 1948	20	2.40	2.35	3	5	4	9	6	15	8	30	10	40
July 15,	10	3.18	2.25	2	4	3	10	5	20	7	30	8	41
July 20,	10	3.12	1.56	3	7	3.5	15	4.5	26	6	42	7	65
July 20,	17	.95	1.19	4	10	8	18	10	32	12	52	15	78
May 9, 1949	10	4.80	2.75	2	4	4	9	6	14	7	22	8	30
May 9,	18	4.20	1.87	4	6	6	10	7	16	8	23	10	47
June 23, 1951	12	3.75	2.45	1	3	2	7	3	14	3	23	4	35
July 22,	19	3.63	1.65	4	6	5	12	6	21	9	34	11	55
July 23,	15	3.80	1.56	1.5	7	2	13	3	21	5	37	6	63
July 23,	15	3.00	1.75	2	3	3	8	5	17	6	30	8	56
Mar. 31, 1952	23	3.08	1.87	4	7	6	12	10	19	10	30	11	38
Averages				3.45	6.72	5.20	12.3	7.03	21.3	8.84	33.7	10.8	50.8

'imes before the peak of the unit graph, being affected by rainfall intensity storm pattern, are not as consistent as times after peak, which are the re- of depletion of detention volumes after cessation of rainfall. The average ɔ of times before to times after the peak, for various percentages of the ;, was determined for each of the watersheds. The results, shown in Fig. 8, cate considerable variation between watersheds. However, since less than of the runoff occurs before the peak, the use of an average curve should introduce serious errors.

'he data and analysis thus far is confined to those areas on which runoff surements are available. The results will have much wider application by ting them to ungaged areas. Points representing the unit graph peak and to peak for rainfall intensities of 1, 2, 3, 4, and 5 in. per hr, taken from curve of Fig. 5, and similar plottings on other watersheds at Edwardsville a 189-acre area near Hamilton, Ohio, are shown on Fig. 9. These curves y to those cases where the period of high intensity occurs late in the storm.

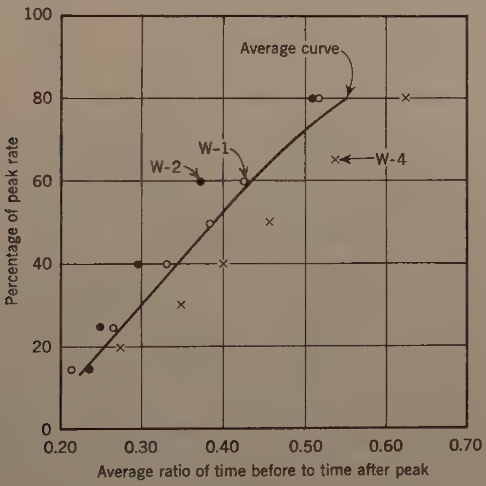
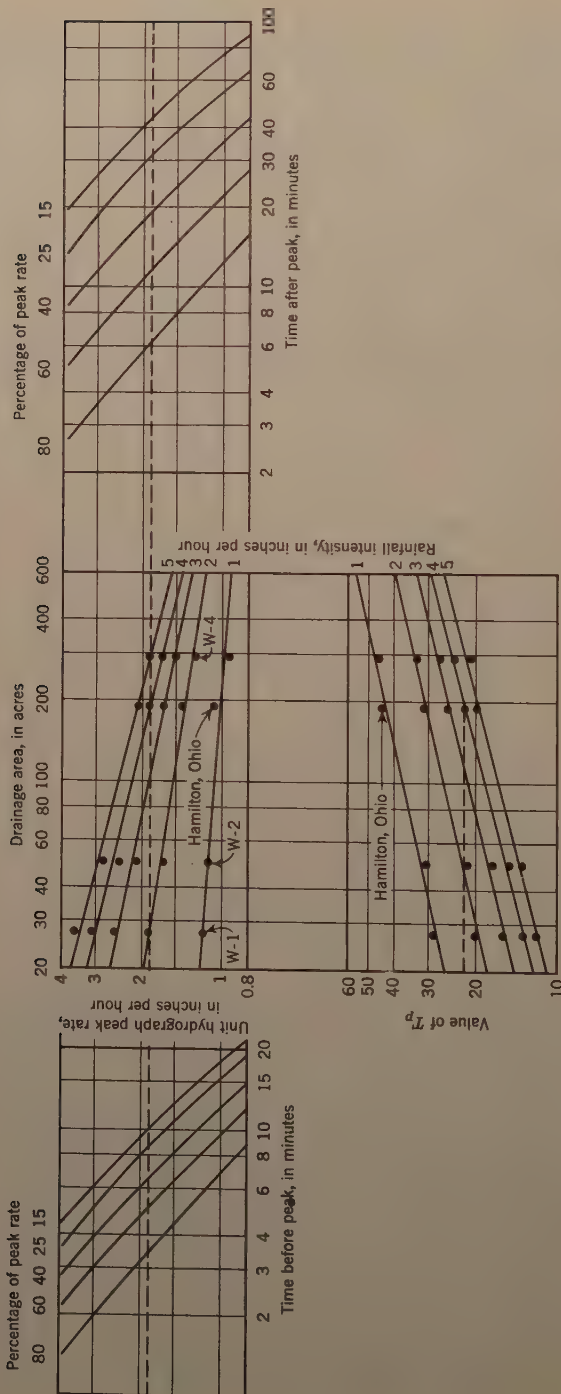


FIG. 8.—RATIO OF TIMES TO PEAK

curves show the effect of size of area on the unit hydrograph peak and time ak. A similar set of curves can be prepared for storms having the in- period at the beginning. The curves for times after the peak, at which ous percentages of the peak rate was reached, were taken from plottings lar to those in Fig. 7. The curves showing times before the peak were de- ined by applying the average ratios from Fig. 8 to the times after the peak. series of synthetic unit hydrographs for a particular size area and differ- ainfall intensities can be obtained from Fig. 9.

ɔ illustrate this, assume a unit hydrograph is required for a rainfall in- ty of 4 in. per hr and a drainage area of 200 acres. To develop the syn- c unit hydrograph, enter Fig. 9 at 200 acres and go vertically to the rain- ntensity of 4 in. per hr, then horizontally from these points to find a peak of 1.85 in. per hr and time to peak of 22 min after the beginning of rain- xcess. Proceed horizontally along the line for a unit graph peak of 1.85 er hr to read times before and after the peak at the percentages shown.



Percentage of peak rate
80 60 40 25 15
Time after peak, in minutes
2 3 4 6 8 10 20 30 40 60 100
Drainage area, in acres
20 30 40 60 80 100 200 400 600
Rainfall intensity, in inches per hour
1 2 3 4 5
Unit hydrograph peak rate, in inches per hour
4 3 2 1 0.8
Hamilton, Ohio
W-1
W-2
W-4
Value of T_p
60 50 40 30 20 10
Percentage of peak rate
80 60 40 25 15
Time before peak, in minutes
1 2 3 4 6 8 10 15 20

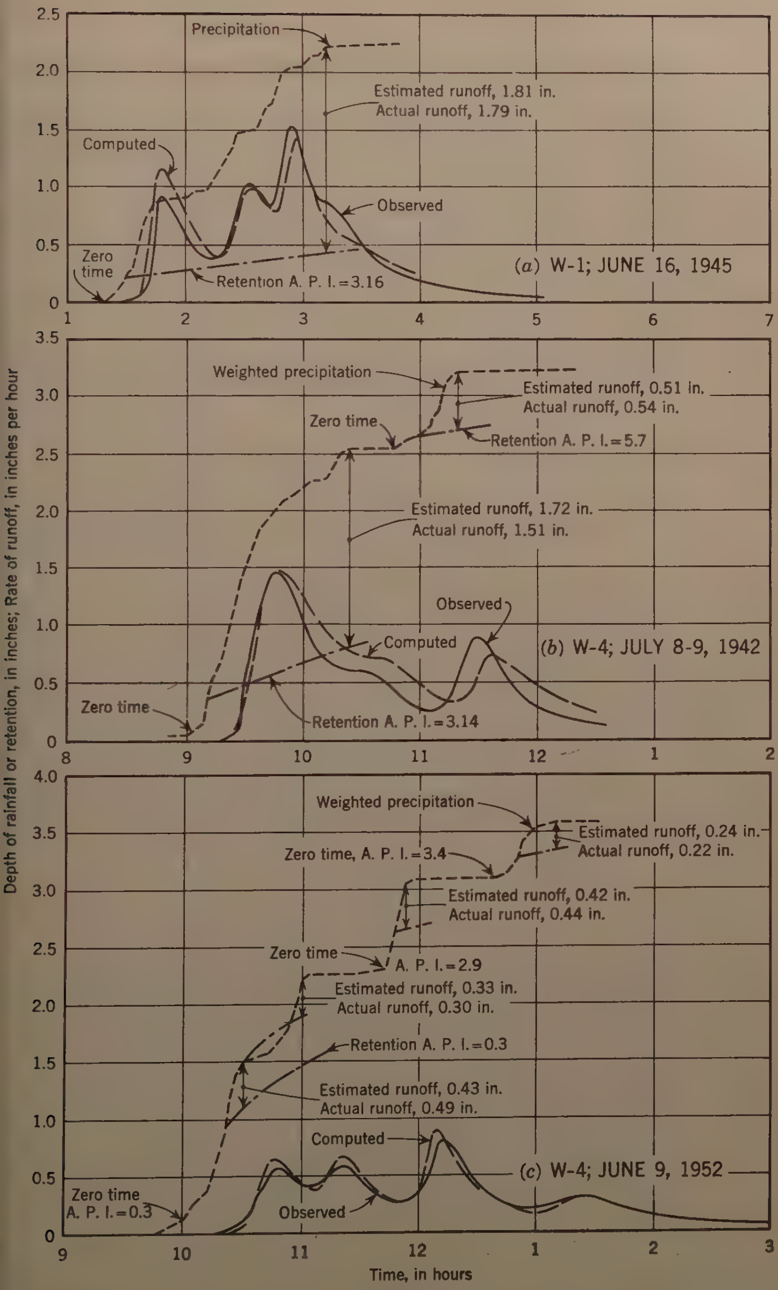


FIG. 10.—COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS

Thus a rate of 60% of the peak would occur 5 min before and 12 min after the peak.

COMPARISON OF COMPUTED AND OBSERVED HYDROGRAPHS

The final step was to apply the time-retention curves and the unit hydrographs previously developed to the actual mass rainfall diagram of a numb

TABLE 4.—COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES

Year	Month	Day	Peak Discharge inches/hour	
			Observed	Computed
1938	March	30	2.29	2.16
	May	27	2.41	2.64
	July	17	1.10	1.00
1939	July	16	.72	.63
	August	17	.76	.88
1940	August	4	.72	.84
1941	September	2	.48	.60
1942	July	8	1.42	1.86
	December	27	.47	.43
1943	May	17	1.19	1.06
		17	.92	1.24
1944	April	22	1.05	1.34
	August	30	.31	.30
1945	June	16	1.55	1.40
1946	August	14	2.57	2.87
		14	1.33	1.41
1947	April	29	1.50	1.60
1948	June	26	.40	.37
	July	20	.53	.50
		25	.83	.77
1949	May	9	1.39	1.56
	June	27	1.19	1.04
		28	2.07	1.83
	August	2	1.00	.92
1950	January	3	.70	.54
		3	.78	.65
1951	June	16	.84	.97
	July	22	.64	.92
		23	.85	.87
		23	1.01	.87
1952	March	31	1.36	1.45
		31	2.05	2.16
	July	2	1.82	1.89
1953	April	23	.51	.73
	June	26	.72	.74
1954	July	4	.92	.63
	September	20	.56	.43
	October	5	.40	.48
		11	.49	.46
		11	.83	.88

of storms on both the 27.2-acre and 290-acre areas to see how the compu hydrographs agreed with the actual record. The method used is illustrated Fig. 10(a) for the storm of June 16, 1945 on the 27.2-acre area (W-1). T mass rainfall diagram shows considerable variation in rainfall intensity dur

excessive part of the storm. The antecedent precipitation index was com-ed and used to select a time-retention curve, to be plotted directly below mass rainfall diagram. The vertical distance between the mass rainfall the retention curve represents the potential runoff to be expected from nfall prior to that time. The difference in these ordinates at selected points es the increment of runoff to be applied to a unit graph selected from Fig. 6.

The first trial was made using a series of 12 min intensities even though breaks in the rainfall diagram indicated considerable variation in intensity ing certain of these periods. Investigations showed that much better agree-nt with the actual hydrograph could be obtained by using variable time in-vals such that the intensity during any interval was more nearly uniform. time intervals selected, as indicated on the mass rainfall diagram, varied nultiples of two and ranged from 4 to 12 min. For each of these intervals, unit graph was selected for which the intensity most nearly matched that he mass rainfall diagram. For example, the intensity from 2:11 to 2:33 was 0 in. per hr and the unit graph used for this period was the one based on the rm of April 17, 1941.

Using the same procedure but applying a series of synthetic unit hydro-phs obtained from Fig. 9 to the storms of July 8-9, 1942 and June 9-10; 1952 vatershed W-4 gave the comparisons shown in Fig. 10(b) and 10(c). Note that retention curves for each of the periods of excess is based on the API at beginning of that period.

The final test of the principles involved was the computation of peak rates one or more of the outstanding storms each year on watershed W-1. Table 4 ws that the difference between observed and computed peak rates is less 1 10% for one-half of the storms. The greatest disagreement between the erved and computed hydrographs was for those storms beginning with a rt period of very high intensity. A slight difference in the position of the e retention curve makes considerable change in the estimated runoff for this ial period. The high rainfall intensity requires use of the unit graph with 1ort, sharp peak, similar to the one derived for the May 27, 1938 storm on ershed W-1, which greatly magnifies any small discrepancy in the estimated off.

CONCLUSIONS

.. There appears to be a good relationship, for this soil type, between ante-ent precipitation and the amount of precipitation retained in a given time.

!. Runoff from the most important storms can be estimated with reason-accuracy from the mass rainfall diagram and time-retention curves.

. These investigations show that on small watersheds, one unit hydrograph not adequately define the shape of the hydrograph derived from a storm of duration.

. For the small watersheds involved in these studies, there appears to be od relationship between rainfall intensity and the peak rate of the unit hy-graph and the time from beginning of excess rainfall to peak rate of runoff.

. A method has been presented for constructing synthetic unit hydrographs drainage areas from 20 acres up to about 500 acres and for different rain-intensities.

. Computed hydrographs show close agreement with the observed record e unit hydrographs used are based on storms having similar rainfall in-ities and distribution patterns.

7. There is need for further research along these lines, particularly to se if similar relations exist for other soil types.

ACKNOWLEDGMENTS

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Proceedings of the American Society of Civil Engineers

DISCUSSION

TWO METHODS TO COMPUTE WATER SURFACE PROFILES[a]

Discussion by S. V. Chitale and K. Arunachalam, F. Paderi

Closure by Joe M. Lara and Kenneth B. Schroeder

V. CHITALE[1] and K. ARUNACHALAM.[2]—The authors are to be congrat-d for the lucid exposition they have given of the step-by-step method ted in the Bureau of Reclamation for backwater computations when both hannel and overbank discharges are required to be considered. Two meth-have been illustrated, Method B having shown to be more accurate of the Two assumptions have been made in the development of method; one is h_f is the same for channel and overbank flow in a given cross section, and ther that velocity head is also the same for both channel and overbank flow cross section. In some cases, these assumptions may not appear to be and their justification would depend on the magnitude of error involved ese assumptions. Application of another step-by-step method without re-se to the above assumptions has been illustrated below. It is contended this method is more general than Method B and its adoption in preference ethod B may be found desirable in specific cases. For comparison, an ple is solved by both the methods; that is, the general method as well as od B (Table 3). Since the data of Red Fox River given by the authors are sufficient to illustrate the more general method advocated below, another —Brahmani River in the State of Assam, India—has been chosen for work-ut comparative results.

eneral Method

ssumption.—The only assumption made is that water level across a cross on is same over the channel and overbank portions. This assumption is non to this as well as Method B.

quations.—Besides those given by the authors, the following additional ions are required to be adopted:

sing the subscripts c and s to distinguish properties of channel and over-portion, at any section,

$$Q_c + Q_s = Q \quad\quad (1)$$

$$Q_c = K\, d_c \left(\frac{h_{f_s}}{L_c}\right)^{\frac{1}{2}} \quad\quad (2)$$

pril, 1959, by Joe M. Lara and Kenneth B. Schroeder.

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$$Q_s = K\, d_s \left(\frac{h_{f_s}}{L_s}\right)^{\frac{1}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

$$\Delta H_c = \Delta H_s = \Delta H \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

$$h\, v_{c_2} - h\, v_{c_1} = \Delta h\, v_c \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

$$h\, v_{s_2} - h\, v_{s_1} = \Delta h\, v_s \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

$$h_{f_c} = \Delta H + \Delta h\, v_c - \text{eddy loss} \quad \ldots\ldots\ldots\ldots\ldots$$

and

$$h_{f_s} = \Delta H + \Delta h\, v_s - \text{eddy loss} \quad \ldots\ldots\ldots\ldots\ldots$$

Design Data.—Backwater computations were carried out for the Brahm River in Assam State, India:

1. A plan map in Fig. 1 of the river shows the location and designation the different cross sections.

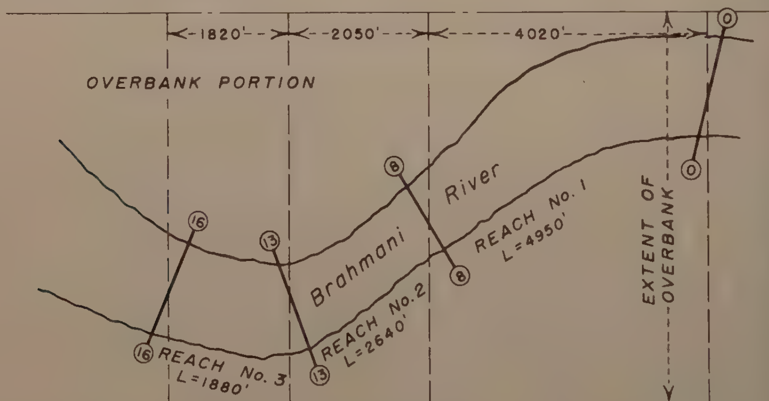

FIG. 1

2. The water surface elevation at Cross Section 0-0 is 682.00.
3. Discharge in the channel portion of cross section 0-0 at water surf El. 682 is 835,000 cfs.
4. Discharge in the overbank portion of cross section 0-0 at water surf El. 682 is 40,000 cfs.
5. "n" = 0.03 for the main channel and "n" = 0.05 for the overbank secti
6. Values of Kd, A, R, etc., for different water levels are calculated given in Table 1.
7. The relationships between water surface elevations and k d_s for differ cross sections are shown graphically in Fig. 2.

Procedure.

Step 1.—To start with, discharges occurring in the overbank and chan portions are known at the end section, that is, at cross section 0-0. In the fi trial for estimating the discharges passing through the two portions at cr section 8-8, the velocity head is ignored, that is, $h_f = \Delta H$. Since ΔH is sa for both the portions, h_f overbank = h_f channel. A trial ΔH is assumed, Col Table 2.

p 2.—The values of K d for the two sections 0-0 and 8-8 are averaged e discharges Q_c and Q_s are computed by Eqs. 2 and 3 and entered in 10 and 17.

p 3.—If the sum of Q_c and Q_s is not equal to the actual total discharge of 0 cfs, the actual discharge is divided in the ratio of Q_c and Q_s obtained 2 and adopted (Cols. 19 and 20) for finding the velocity head in the next

TABLE 1.—BRAHMANI RIVER

Elev.	A	P	R	K d	Reach length	h v	Bankful Stage
(a) Properties of Channel Portion							
680.00	63,408	1,413.0	44.87	39,665,756		2.8174	
682.00	66,172	1,415.6	46.75	42,509,164		2.5880	
684.00	68,936	1,418.0	48.61	45,479,809	4,950	2.3837	
683.30	48,230	1,167.8	41.30	28,546,541		4.8700	
686.00	51,330	1,171.6	43.81	31,601,720		4.3000	
688.00	53,626	1,174.6	45.65	33,944,904	2,640	3.9390	
686.00	60,307	1,265.8	47.64	39,219,254		3.1150	
688.00	62,759	1,268.4	49.48	41,901,816		2.8760	
690.00	65,211	1,271.4	51.29	44,604,793	1,980	2.6640	
686.20	73,700	1,638.4	44.98	46,177,066		2.0860	
688.00	76,920	1,640.8	46.88	49,566,117		1.9150	
690.00	80,104	1,643.2	48.77	52,990,611		1.7660	
(b) Properties of Overbank Portion							
680.00	13,060	4,366	3	815,000			
682.00	21,800	4,370	5	1,910,000			
684.00	30,520	4,374	7	3,330,000	4,950		R.L. 677.00
682.00	19,520	4,888	4	1,470,000			
684.00	29,280	4,892	6	2,870,000			
686.00	39,040	4,896	8	4,630,000	2,640		R.L. 678.00
684.00	22,460	4,502	5	1,950,000			
686.00	31,444	4,506	7	3,430,000			
688.00	40,428	4,510	9	5,420,000	1,980		R.L. 679.00
684.00	17,816	4,462	4	1,340,000			
686.00	26,724	4,466	6	2,620,000			
688.00	35,632	4,470	8	4,240,000			R.L. 680.00

4.—Again a trial ΔH is assumed. This is added to the water surface n at cross section 0-0 to get water surface elevation at cross section

5.—The velocity heads $h\,v_c$ and $h\,v_s$ using the above water surface ele- re calculated by using the discharges obtained in step 3 (Cols. 4 and 11).

TABLE 2.—BACKWATER COMPUTATIONS FOR BRAHMANI RIVER U

Cross section	Trial ΔH	Water surface elevation	$\frac{V^2}{2g} = hv_c$	Δhv	Eddy loss	hf_c	Kd x 10^6	Kd_{avg} x 10^6	$Kd \sqrt{h_f} + \sqrt{L} = Q_c$ x 10^5 cfs	$\frac{V^2}{2g} = hv$	Δ
1	2	3	4	5	6	7	8	9	10	11	
0-0		682.00	2.46				42.51		8.35	0.052	
8-8	1.50	683.50				1.50	29.00	35.75	6.20		
	1.50	683.50	4.50	2.04	1.02	2.52	29.00	35.75	8.05	0.065	0.
	1.60	683.60	4.50	2.04	1.02	2.62	29.25	35.87	8.30	0.065	0.
13-13	2.00	685.6				2.00	39.25	34.25	9.60		
	3.40	687.0	2.56	-1.94	0.19	1.27	41.00	35.13	7.75	0.087	0.
	3.20	686.8	2.30	-2.20	0.22	0.78	40.75	35.00	6.05	0.370	0.
	3.60	687.2	2.06	-2.44	0.24	0.92	41.20	35.23	6.60	0.30	0.
	3.80	687.4	2.05	-2.45	0.24	1.11	41.50	35.38	7.20	0.30	0.
16-16	1.50	688.9				1.50	50.80	46.15	12.70		
	1.50	688.9	1.65	-0.40	0.04	1.06	50.80	46.15	10.70	0.056	-0.
	1.10	688.5	1.655	-0.395	0.04	0.665	50.20	45.85	8.40	0.06	-0.
	1.00	688.4	1.66	-0.39	0.04	0.57	50.00	45.15	7.80	0.06	-0.

TABLE 3.—BACKWATER COMPUTATIONS FOR BRAHMANI RI

Section	Reach length	Water surface elevation	Area	Kd x 10^6	$\frac{Kd}{\sqrt{L}}$ x 10^6	$Q + \Sigma \frac{Kd}{\sqrt{L}} = h_f$
1	2	3	4	5	6	7
0-0		682.00	66,172	42.51	0.600	
			21,800	1.91	0.030	1.92
8-8	4,950	682.75	46,538	28.25	0.400	
	4,020		23,120	2.00	0.032	4.05
Trial II		684.00	49,034	29.50	0.420	
			29,280	2.90	0.064	3.27
Trial III		684.15	49,140	29.27	0.420	
			29,770	2.95	0.065	3.22
13-13	2,640	686.00	60,307	39.22	0.765	
	2,050		31,444	3.43	0.076	1.04
Trail II		687.00	61,533	41.00	0.800	
			35.932	4.30	0.095	0.95
Trial III		687.30	61,899	41.25	0.805	
			37,278	4.55	0.100	0.935
Trial IV		687.34	61,990	41.25	0.805	
			37,291	4.55	0.100	0.935
16-16	1,980	689.00	78,512	51.00	1.140	
	1,820		40,086	4.85	0.116	0.485
Trial II		688.80	78,352	50.75	1.140	
			39,186	4.70	0.110	0.490

RAL METHOD SUGGESTED BY S. V. CHITALE AND K. ARUNACHALAM

h_{f_s}	Kd x 10^6	Kd_{avg} x 10^6	Kd $\sqrt{h_f}$ + $\overline{L}$ = Q_s x 10^5 cfs	$\frac{Q}{Q_c + Q_s}$	Q_c x 10^5 cfs	Q_s x 10^5 cfs	$\left(\frac{Q}{Q_c + Q_s}\right)^2$ h_{f_c}
14	15	16	17	18	19	20	21
	1.90						
1.50	2.50	2.20	0.40	1.32	8.20	0.55	Velocity head neglected
1.507	2.50	2.20	0.425	1.03	8.30	0.45	2.67
1.607	2.55	2.22	0.44	1.00	8.30	0.45	1
2.00	3.10	2.82	0.875	0.82	7.9	0.85	Velocity head neglected
3.41	4.30	3.40	1.39	0.96	7.45	1.30	1.17
3.35	4.10	3.30	1.32	1.18	7.15	1.60	1.09
3.72	4.50	3.50	1.49	1.08	7.15	1.60	1.18
3.92	4.62	3.56	1.54	1	7.20	1.55	1
1.50	4.80	4.71	1.35	0.622	7.9	0.85	Velocity head neglected
1.23	4.80	4.71	1.23	0.735	7.85	0.90	0.57
0.836	4.50	4.56	0.98	0.935	7.85	0.90	0.58
0.826	4.43	4.52	0.965	1.00	7.80	0.96	0.57

G METHOD B AS USED BY JOE M. LARA AND KENNETH B. SCHROEDER

$\sqrt{L})(\sqrt{h_f}) = Q$	V	V^2Q x 10^5	hv	$hv_1 - hv_2$	Eddy loss	Mean h_f	Total loss	H
8	9	10	11	12	13	14	15	16
8.35	12.6	1,320.00						
0.40	1.83	1.34	2.37					
8.08	17.00	2,340.00						
0.67	2.90	5.60	4.17	1.80	0.90	2.99	3.89	2.09
7.60	15.40	1,790.00						
1.15	3.93	17.70	3.22	0.85	0.43	2.59	3.02	2.17
7.57	15.35	1,790.00						
1.18	3.94	17.70	3.22	0.85	0.43	2.57	3.00	2.15
7.95	13.20	1,380.00						
0.80	2.55	5.20	2.46	-0.76	0.08	2.13	2.22	2.98
7.78	12.60	1,230.00						
0.97	2.70	7.10	2.20	-1.02	0.10	2.09	2.19	3.21
7.80	12.60	1,230.00						
0.95	2.55	7.10	2.20	-1.02	0.10	2.07	2.17	3.19
7.80	12.60	1,230.00						
0.95	2.55	7.10	2.20	-1.02	0.10	2.07	2.17	3.19
8.00	10.20	8.20						
0.75	1.87	2.62	1.52	0.68	0.07	0.71	0.78	1.46
8.00	10.40	8.65						
0.75	1.91	2.73	1.54	0.66	0.07	0.713	0.783	1.443

Step 6.—The terms $\Delta h\ v_c$ and $\Delta h\ v_s$ are calculated by Eqs. 5 and 6 entered in Cols. 5 and 12.

Step 7.—Eddy loss is assumed at 50% of $\Delta h\ v$ for retarded flow and 10 $\Delta h\ v$ for accelerated flow; Cols. 6 and 13.

Step 8.—The values of h_{f_c} and h_{f_s} are then computed by Eqs. 7 and 8 entered in Cols. 7 and 14.

Step 9.—The value of K d for the two sections 0-0 and 8-8 are averaged discharges Q_c and Q_s are computed.

Step 10.—The discharges obtained in Step 9 still do not satisfy Eq. 1. get an idea of h_f to be used in the next step, the following equation is u

$$Q^2 = K\ d^2\ \frac{h_f}{L} \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

$$\left(\frac{Q_{actual}}{Q_{computed}}\right)^2 = \frac{h_f}{h_{f_{computed}}}, \text{ approximately } \ldots\ldots$$

Solving for h_f:

$$h_f = \left(\frac{Q_{actual}}{Q_{computed}}\right)^2 h_{f_{computed}} \quad \ldots\ldots\ldots\ldots$$

This assumes that K d remains constant for small change in water sur elevations.

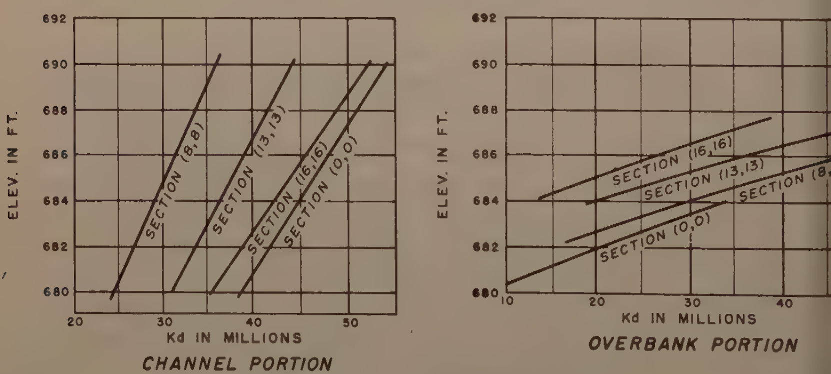

FIG. 2

As the flow in the channel usually predominates, h_f used in Eq. 11 is for channel portion. The terms Q_{actual} and $Q_{computed}$ are the total charges. The value of h_f obtained by Eq. 11 is entered in Col. 21 for comp son with the assumed h_f, Col. 7 so as to give an idea of ΔH to be used in next step.

Step 11.—As we proceed, the discharges Q_s and Q_c approach the co value. This procedure is repeated until Eq. 1 is satisfied. The same pr dure is used to compute the upstream water surface elevations for cross tions 13-13 and 16-16.

. PADERI.[3]—The writer has had occasion to compute water surface pro- for flow in open channels. He particularly appreciated the clear and ac- te exposition on the two methods of finite differences derived and is pleased them for several reasons, among them being:

The development of Methods A and B is based on a profound and exten- analysis of the various physical phenomena involved and the relative ef- of each in the theoretical analysis.

The authors point out that practical judgment, in addition to theoretical sis, also plays an important part in the evaluation of these phenomena.

The writer noted in this paper several ideas he developed when dealing the same subject.[4,5]

writer has nothing to add to the first two above statements, but some ex- tion of the third follows:

A fundamental element of computing by finite differences as presented e authors is the hypothesis of linear variation of the function S_f on each vision, specifically its equivalent

$$\frac{Q^2}{A^2 \chi R}$$

e χ, defined by Manning has a value in metric units:

$$\chi = \frac{R^{\frac{1}{6}}}{n} \quad \ldots\ldots (12)$$

n American units:

$$\chi = \frac{1.486\, R^{\frac{1}{6}}}{n} \quad \ldots\ldots (13)$$

in metric units:

$$S_f = \left(\frac{Q\,n}{A\,R^{\frac{2}{3}}}\right)^2 \quad \ldots\ldots (14)$$

n American units:

$$S_f = \left(\frac{Q\,n}{1.486\,A\,R^{\frac{2}{3}}}\right)^2 \quad \ldots\ldots (15)$$

inear variation hypothesis, on each subdivision, of the ratio

$$\frac{Q^2}{A^2 \chi^2 R}$$

sst. Prof., Univ. of Pisa, Italy.

Sul moto permanente delle correnti a superficie libera," by F. Paderi, Richerche gneria, Roma, marzo-aprile, 1939.

Sul profili di rigurgito di correnti lente," Stab. Tip. gia Chiari, Succ. C. Mori., e, 1946, e Bollettino Tecnico Ingegneridi Firenze, aprile-maggio, 1948.

(not of each element such as A or R, as used in previous studies) was alr stated, to the best of the writer's knowledge, for the first time in the writ report of July, 1946, and presented at the Commission for the Professor (Chair of Hydraulics: D. M. June 28, 1946, and August 6, 1946).

The above-mentioned hypothesis was used in the same report (applying method of finite differences) for the computation of water surface profiles subcritical flow in cylindrical channels. An example was shown in Section the report applicable to specific conditions.

The writer appreciates the authors' confirmation of the validity and pr ticality of the forementioned hypothesis, implicit in the evaluation of h_f, each subdivision, with

$$h_f = (L \div 2)\left(S_{f_1} + S_{f_2}\right) \ldots\ldots\ldots\ldots\ldots\ldots$$

2. A second element as important to practical hydraulic analysis as the discussed previously concerns the proper introduction in the computation a simplified and practical concept of eddy loss incident to the conversio kinetic to static head and vice versa in open channel flow. This is usually c sidered as some portion of the change in velocity heads.

In previous work[4,5] (dealing mostly with subcritical flow) the writer veloped this same concept which was discussed in general terms. Base previous studies of this concept, the writer introduced in his conclus another coefficient, ρ (analogous to the authors' coefficient C) combined the coefficient X of uniform flow to evaluate the friction head and eddy los holding to the true phenomena. Regarding the choice of the ρ value whe constant or variable according to the type and condition of flow, the wr pointed out evidence for the necessity of conducting further research in part lar the carrying out of proper experimental tests. These tests should be gea to attain a better evaluation, both quantitatively and qualitatively, of the ρ efficient.

The writer also explored the results of particular values of ρ, usefu practical applications ranging from zero to one and reciprocal departure the water surface profiles. For specific conditions, interesting evaluation the coefficient ρ or C, also designated as $-\rho$ or k, were obtained in later st ies[6,7] on this important subject.

Regarding these studies and those of the authors, it is important to note minute numerical departures are not bound to a particular single set of co tions which can be said of the general agreement of the fundamental phys concepts involved. Under certain circumstances the value of ρ, C, etc., c also possibly be accounted for in the value of X itself which has n as the va able factor.

It is well to note also that with the introduction in Bernoulli's energy eq tion of the eddy losses affected by the coefficient ρ, C, $-\rho$, or k, there rema in the said equation a single multiplier $1 - \rho$ (or $1 - C$, $1 + \rho$, etc.) of the ference in velocity heads. Bernoulli's equation as quoted in paper can be w ten as follows considering only the eddy losses mentioned above:

$$Z_2 + d_2 = Z_1 + d_1 + h_f + (1 - C)\left(h_{v_1} - h_{v_2}\right) \ \ldots\ldots\ldots$$

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[6] "A duzzasztási görbek számitasara ajanlott módszerek hidromechanikai összel onlitása," by Gy. Kovacs, Vizügyi Közlemények, N. I. Budapest, 1952, pp. 84-92.

[7] "Etude et tracé des écoulements permanents en canaux et rivierés," by R. Sil Dunod, Paris, 1954, p. 13.

ad of the factor (1 - C), a value of (1 + C) for this coefficient can be used, an algebraic change in sign is noted. Based on the 100% value reported e authors for retarded flow conditions, the factor is resolved as follows:

$$1 - \rho = 1 - C = 1 + \rho = 1 - k = 0 \quad (18)$$

authors are commended on their calling attention to the numerical value which can change to fit the conditions observed and its assigned value is o the judgment of the hydraulic engineer.

OE M. LARA[8] and KENNETH B. SCHROEDER.[9]—The writers appreciate nteresting discussions submitted on the methods of computing water sur- profiles.

lessrs. Golding, Anand, and Silvester introduced the idea of expressing eyance as K equal to 1.486 A $R^{2/3}$. Adjustment of the K_d-values used by riters can be readily facilitated in cases involving variations in "n." This articularly adaptable to conditions where the water surface profiles have observed for a series of discharges which results in a definite "n"-stage ionship. The writers found this idea very helpful in subsequent studies.

Ir. Golding's additional suggestion of replacing Col. 5 of Table 2 with two mns is also useful when applying the preceding analysis. He suggests spe- ally tabulating the values of K = 1.486 A $R^{2/3}$ in one column and K_d = K/n e other. Mr. Golding's graph plotting the conveyance "k" vs. depth indeed d be helpful in the case of an infinitely wide flood plain. The graphs pre- d by Mr. Silvester in nomographical form, wherein the solution of the tion, Q = (K/n) $(S^{1/2})$ is included for various prismatic channels also d be helpful in certain cases.

he refinement of using different flow travel paths was questioned by srs. Golding and Peterson of Method B. This method has not been tried in cient cases by the writers to arrive at a more conclusive appraisal. In ew cases tried, it was also found by the writers that computed elevations not significantly affected. The method merits though from the standpoint nsidering the existent hydraulic conditions realistically, yet conforming andard theoretical analysis.

he writers acknowledge with interest the work carried out by Mr. Golding e study wherein the "n" values were increased based on observed data. oncluded that when the deflection angle exceeded 45 degrees, the "n" value fiably could be increased. Although Mr. Anand referred to the formula, loss = coeff. x 1/2 $\left(h_{v1} + h_{v2}\right)$, he did not offer any suggestions or guides signing a numerical value for the coefficient. This, as with other hydraulic mptions, undoubtedly is left to the judgment of the engineer.

essrs. Anand and Peterson offer further discussion of the eddy loss analy- From actual hydraulic measurements, Mr. Anand verified his selection dy loss coefficients which probably supports his recommending that eddy s be neglected for converging reaches but a 50% correction used for a rging reach. Mr. Peterson, on the other hand, suggests a 10% correction onverging reaches and from zero to 100% for diverging reaches.

r. Peterson's presentation on developing the coefficient α_1, and his analy- f the continuity equation resulting in the formula, Q = K_{dg} $S_g^{1/2}$, are of emic interest. His example of computing the water surface profiles is

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Hydr. Engr., Bur. of Reclamation, Denver, Colo.
Hydr. Engr., Bur. of Reclamation, Denver, Colo.

another approach incorporating the application of this α correction and us the geometric mean of the K_ds. The results, however, are quite different fr those of the writers. About a 50% error in discharges is indicated.

It is to be remembered that in the final analysis of any method, regardl of the mathematics involved in satisfying Bernoulli's energy equation, the co puted results carry only the same degree of accuracy as that of the obser field data. The moment eddy losses and corrections for velocity head chan are introduced in any of these computations, the element of judgment by individual engineer plays a significant role. The writers believe, therefo that greater emphasis be placed upon gathering field data which are jud representative of the hydraulic conditions prevalent in the reach under inves gation. Whatever mathematical analysis is applied to these data is at the d cretion of the engineer. Undoubtedly many of us have our "pet" methods.

The following comments apply to the discussion submitted by Messrs. Chit and Arunachalam:

It was not the writers' intention to show Method B was more accurate t Method A. The primary reason for presenting the two methods was to sh computational procedures covering situations where the flow lengths betw sections were different in the main channel and overbank area. However, mentioned previously, the results, applying Method A and using only the f length of the main channel, do not vary significantly from those using Met B as applied to various problems thus far.

There are several comments regarding the supporting data that require planation or clarification as listed below:

a. There appears to be an error in the reach lengths quoted in Table They are listed as being identical for the main channel and overbank porti Reach lengths shown in Fig. 1 do not seem representative of the flow path ta by the water in the overbank areas. It seems more reasonable that the f would follow more of sinuous path somewhat parallel to the main channel shorter.

b. It would be interesting to know how the hydraulic properties of e cross section (Figs. 1 and 2, Table 1) for the overbank portion were det mined. The important question is: What happens when the bankfull stage reached and then surpassed? Normally, the cross section is laid out in field in a straight line extending to the limit of the overbank portion. If t were done in the example, Sections 13-13 and 16-16 would intersect in overbank area imposing some complexity in the computation of the hydrau characteristics.

c. A lesson in significant digits is to be gained in the tabulation of the va ous hydraulic characteristics listed in Table 1. In general, these can be h to three digits yet be compatible and representative of the gathered field da A look at the tabulated data shows that they range from one place in R of overbank portion to eight places in the K d values for the main channel. The is some inconsistency.

d. Velocity head (hv) values appear in Table 1 for all elevations of the m channel of the four cross sections. No mention is made as to how or why the values were tabulated. A check shows they were all computed on a basis o discharge equal to about 855,000 cfs.

e. In Table 1 there is also listed values for a term R.L. at each secti It is presumed this means "reference level." However, to list them as ba full stage is another unexplainable item.

n exception to some of the above comments, Messrs. Chitale's and Aruna-lam's method is judged meritorious and appears to be a worthwhile proce-e. The results of the discussers' and writers' procedures are plotted in . 3 to show comparative differences.

Mr. Paderi presents an interesting discussion of the results he attained in studies involving finite differences. The writers have not had the oppor-ty to review any of the literature quoted by Mr. Paderi. It is noted, how-r, that most of his experiments and studies dealt with subcritical flow which

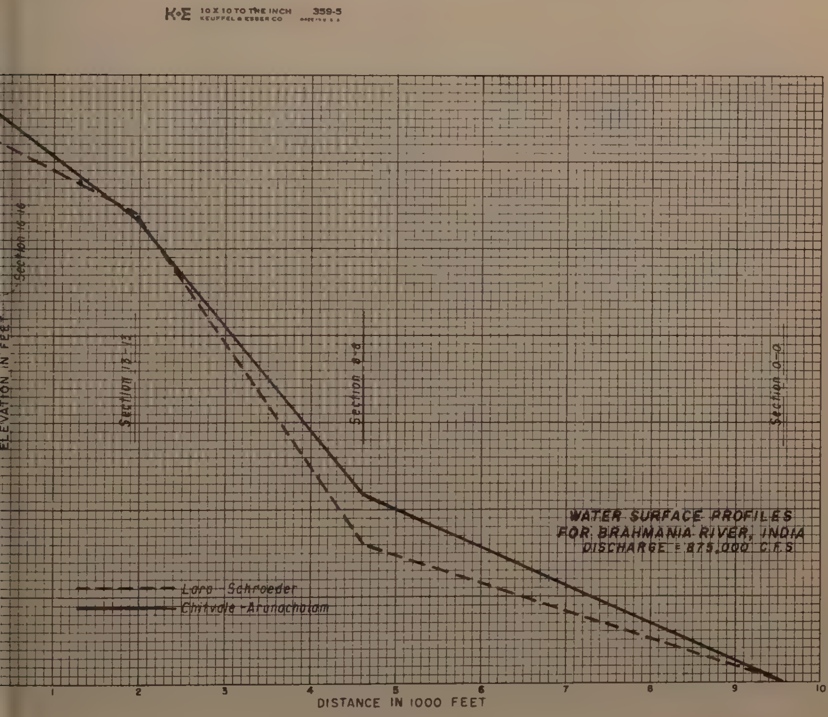

FIG. 3

writers believe he meant flows in the "shooting" range or at supercritical cities. Mr. Paderi's comments relative to the coefficient assigned for ing eddy loss corrections are covered by previous discussors. The writ-appreciate Mr. Paderi's remarks on the opinion he shares regarding the that there is still a lot of responsibility riding on the engineer's shoulders e selection of these coefficients and it might be added that this applies also ne assignment of "n" values for use in Manning's formula.

FLOODS OF THE FLORIDA EVERGLADES[a]

Closure by Edwin W. Eden, Jr.

EDWIN W. EDEN, JR.[1]—Any discussion of the Everglades is incomplete ithout information on the Seminoles—its natural inhabitants—and their role the development of this unique area. There can be no doubt that the Semi-oles have learned to live in the extremely wet, hot, and humid environment. he flood problems arose when development by the "white man" required re-ef from the extremely frequent and long-duration flooding. Mr. Burnet's scussion is greatly appreciated as it provides background on the Seminoles an extremely interesting phase—the development of the Everglades.

[a] June, 1959, by Edwin W. Eden, Jr.

[1] Chf., Planning and Reports Branch, Engrg. Div., U.S. Army Engrg. Dist., Jackson-lle Corps of Engrs., Jacksonville, Fla.

the use of Eq. 8 for the solution of K. Since this is an analytical expressio the value of K can be solved to any desired accuracy. However, Eq. 8 is rather involved mathematical expression, a solution for K is not an easy matte It is believed that the graphical methods will provide a much easier and fast solution for those who are not mathematically inclined. For the sake of greate accuracy in using the graphical method, a probability paper of large sca should be used.

The study made by Mr. Sammons on the accuracy of the graphical metho is very interesting. When either the original variate or the log-transform variate is used, however, the result should be theoretically the same.[2] A difference between the two approaches should resort to the personal error i volved in the graphical construction.

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[2] Closure to "The Probability Law and Its Engineering Applications," by Ven Te Cho Proceedings, ASCE, Vol. 82, 1956, pp. 6-7.

DETERMINATION OF HYDROLOGIC FREQUENCY FACTOR[a]

Closure by Ven Te Chow

VEN TE CHOW,[1] M. ASCE.—The writer appreciates the constructive and eresting discussions by Messrs. Weiss and Sammons. Both gentlemen sugt the methods using the transformed variate. Mr. Weiss suggests an anacal approach, while Mr. Sammons suggests a graphical approach.

It is believed that each method has its own merits and it should best apply a case as required by the circumstances under consideration. The analytical thod requires only the direct solution of a mathematical equation. It can, refore, produce results to any desired accuracy, but the solution of the imcit equation for the frequency factor is not an easy matter. The graphical thods replace the mathematical solution of an equation by the graphical conuction. Their accuracy depends, of course, on the scale of the probability er used in the graphical construction. The use of an ordinary probability er should be satisfactory for practical purposes. For a theoretical study ich requires high degree of accuracy, the analytical method is recommended.

When y is normally distributed, its mean $\bar{y}$ is equal to its median M_y or

$$\bar{y} = M_y \qquad (1)$$

ich is the value of y at a probability of "50%-of-time." Since M is the dian of x, it follows

$$M_y = \ln M \qquad (2)$$

om Eqs. 1 and 2,

$$\bar{y} = \ln M \qquad (3)$$

$$M = \exp(\bar{y}) \qquad (4)$$

ostituting this expression for M in Eq. 6 in the original paper and solving $\bar{x}$,

$$\bar{x} = \exp\left(\bar{y} + \sigma_y^2/2\right) \qquad (5)$$

Mr. Weiss uses this equation to derive an expression for a reduced varie t. The latter is essentially a frequency factor in terms of the variable y. om Mr. Weiss' Eqs. 3 and 4, the following may be written:

$$y/\bar{y} = 1 + C_v' K_y \qquad (6)$$

ere

$$C_v' = y/\bar{y} \qquad (7)$$

d

$$K_y = t = \sigma y/2 + \left(1/\sigma_y\right) \ln\left(1 + C_V K\right) \qquad (8)$$

mparing with Eq. 1 in the paper, it may be noted that Eq. 6 is an application Chow's general formula to the transformed variable y. Mr. Weiss suggests

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[a] July, 1959, by Ven Te Chow.

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NERALIZED DISTRIBUTION NETWORK HEAD LOSS CHARACTERISTICS[a]

Discussion by G. C. Anderson, J. V. Radziul, and P. Celenza

G. C. ANDERSON,[16] M. ASCE.—The author has recognized that the method proportional loads is not applicable for determining the ability of a distri-ion system to supply fire-flow demands. After an arterial system, capable satisfactorily supplying the expected domestic and industrial consumption the design period, has been determined, it will be necessary to consider ividually all possible fire-flow demands on the system. This should be e considering consumption at the maximum daily rate (the average rate of sumption on the maximum day) for the design period. The fire flow will e to be available from the arterial system at sufficient pressure so that it y be delivered through the secondary feeders and the minor distributers to desired fire location with a residual pressure of 20 psi.

J. V. RADZIUL,[17] and P. CELENZA.[18]—The author is to be congratulated his clear, concise documentation and presentation of a new method of cal-ation for the determination of overall system head losses through a wide ge of demand and equalizing storage rates utilizing a minimum of network lyses. His step-by-step solution of actual sample problems should be of istance not only to the hydraulic engineer in the design office but to the er works operator as well.

It should be noted that some water distribution network designers may not sider that the functional relations of Eq. 1 represent an entirely new con-t,[19] since various types of approximation techniques somewhat similar in ure have been used for years, in network analyses, to obtain crude esti-tes of source to point system head losses under varying conditions of sys- input without complete network calculations. However, this paper for the t time has proven that the functional relations of Eq. 1 can be satisfied only er proportional loading. Eq. 2 is a new and entirely original contribution he published literature. Application of the principles embodied in Eqs. 1 2 will improve water distribution network design and water system opera-.

The writers believe that these procedures, in addition to augmenting design nputation techniques, provide a simple practical tool which can be readily lied by most water works operators or engineers in appraising the limita-s of an existing water distribution system from either an engineering or rational viewpoint. The author's contribution is indeed timely and valuable

a January, 1960, by M. B. McPherson.
16 Supervisory Engr., Natl. Bd. of Fire Underwriters, New York, N. Y.
17 Distribution Design Engr., Philadelphia Water Dept., Philadelphia, Pa.
18 Asst. Distribution Design Engr., Philadelphia Water Dept., Phila., Pa.
19 "Flow and Loss of Head in Distribution Systems," by J. J. Doland, Journal, A. W. A., Vol. 35, No. 234, March, 1943.

when one considers that practically all of the approximately 18,000 water d tribution systems in these United States are experimenting "growing pains" are in the throes of substantial rehabilitation (Of the 18,000, about 300 major systems.).

With these procedures as an aid, the evaluation of an existing water dist bution system can be accomplished without considerable expenditure of ti or money for equipment. Good water works practice incorporates a yearly bi-yearly inspection of fire hydrants. Inspection procedure could easily amended to include a zero-flow pressure reading and the time of recordi This information, together with the elevation of the fire hydrant and norma available pumping station and storage operating records, will establish su cient data for the use of the author's equations. Computations can then made to determine head losses at a given point for any range of system dem and system input rates.

Operators are confronted with the urgent need for pressure data, parti larly for the peak hour of maximum day consumption. This field data is di cult to obtain due to timing, coincidence of peak consumption loads and ma mum operation and construction work loads, personnel and equipment limi tions, and the impossibility of saturating any sizeable area with press gages. The need is now obviated, since head losses for any given point car calculated directly for any system demand rate.

Hydraulic grade contour maps for different levels of consumption car made for an existing system without recourse to a computer study or elabo hand calculations. The hydraulic grade contours superposed on a pipe syst diagram would provide a graphic picture of the system, indicating the stre or weakness of the various areas. The study undoubtedly would confirm c tain known low pressure locations and would probably reveal other conditi which are not readily apparent. Comparison of the slopes of hydraulic gradie and pipe sizes would indicate those supply lines in which corresponding fl are much less than potential capacities. Overloaded supply lines would be dicated in the same manner. Those locations for which the data is obviou inconsistent with calculated values obtained via Eqs. 1 or 2 should bear furt investigation in the field. The writers' experience indicates that inconsist cies can usually be traced to the influence of adverse local field abnormalit such as closed or partially closed valves, open boundary control valves tween pressure districts, or local non-proportional loads (severe fluctua of a sizeable industrial or commercial drawoff).

In the author's discussion on proportional load characteristics wit equalizing storage, he states with reference to Fig. 4 that "correlation is g except for the "Southwest" station. Part of the scatter of field data, but probability of inexact proportionality of local loads must be considered at l partially responsible." The writers performed the network analyzer de runs for the Belmont Gravity District shown in Fig. 4 and are in full agr ment with these statements. The "southwest" main is the principal fee serving a large industrial water consumer located approximately 5.9 air miles from the filtered water basin. During the period of field testing rep sented in Fig. 4, pump operation at this industrial plant caused pressure f tuations of 3 to 10 psi; the resulting water hammer when the pump was sta or stopped produced a local short-time pressure fluctuation of up to 38 psi damped effects transmitted clear back to the filtered water basin. This s ation was remedied by having the pump draw directly from an existing sto reservoir at the plant, filling the reservoir from the system. The writers

ident that closer correlation would not be obtained with new field measure-ts at the "Southwest" point since "calculated" head losses to the plant (fed 6-in. main shown at bottom of Fig. 3) are in good agreement with current ational data telemetered to the Load Control Center, which includes a con-ous record of pressure at the plant.

ig. 6A shows the proposed Belmont High Service District for the near fu- and incorporates proposed improvements consisting of a 20-in. reinforc-nain and an elevated tank. The existing 1960 district is a closed pumping em without storage. Current data for this district covering a wide range ow data selected at random from the log sheets of the City of Philadelphia's l Control Center appear in Table 5. Elevation of the station discharge

TABLE 5.—BELMONT HIGH SERVICE DISTRICT (1960)

Date-Time	P_B, Discharge Pressure in psi	$Q_d = Q_p$, in mgd	Gage Pressure at Point X, in psi	Σh_1, in feet, from P_B to Point X		
				Measured (Field Data)	Calculated	
					m = 1.85	m = 2.00
3/14/60 0000	53.8	6.4	64.5	17.0	17.3	16.8
3/14/60 1830	54.0	10.0	54.6	40.3	39.6	40.9
3/14/60 2330	53.6	8.0	63.0	26.2[a]	26.2	26.2
3/20/60 1630	58.4	8.5	64.0	28.8	29.3	29.6
4/2/60 0000	53.5	6.2	66.0	12.8	16.3	15.7
4/2/60 0200	48.4	5.0	62.3	9.6	10.9	10.2
4/2/60 1300	54.0	9.2	57.2	34.3	33.9	34.7
4/3/60 0200	51.9	4.1	66.6	7.8	7.6	6.9
4/3/60 2330	55.2	7.2	63.4	22.7	21.6	21.2

Used to determine K_1 for "calculated" head losses. For m = 1.85, K_1 = 0.559. For 2.00, K_1 = 0.409.

ssure gage at P_B is 258.3 ft. The pressure gage at Point X is located on -in. main, very near the site of the future elevated storage tank shown in 6A. This 8-in. main comprises part of the distribution grid of a small l area supported by the 12-in. main shown in Fig. 6A and an old 8-in. and ld 12-in. feeder in the vicinity of the future 20-in. main. Elevation of the l pressure gage at Point X is 216.6 ft. Item No. 3 in Table 5 was arbitrarily cted to determine K_1. Since $Q_d = Q_p$ at P_B, Eq. 1 applies. For $\Sigma h_1 = 26.2$ ith m taken as 1.85, $K_1 = 0.559$; with m taken as 2.00, $K_1 = 0.409$. The culated" head losses from P_B to Point X, when using either m = 1.85 or , compare quite favorably with the "measured." With the exception of

Item No. 5 in Table 5 "calculated" values are within 1/2 lb of the "measur values or indeed well within the normal combined probably error of the gag The Belmont High Service District is predominately a residential system v relatively minor industrial usage. The close agreement of the results v actual values, for this system at least, lends further validity to the use of p portional loading as a design assumption. Inconsistencies between an ear field survey and calculations made at that time by the author led to the loca

TABLE 6.—TORRESDALE HIGH SERVICE AND FOX CHASE BOOSTER DISTRICT (1960)

Item No.	Date-Time	P_T, Discharge Pressure in psi	Gage Pressure at Point Y, in psi	P_T, in mgd	Oak Lane Station, mgd	Standpipes A, mgd	$\frac{Q_{PT}}{Q_d}$	Σh, in feet, from I to Point Y: Measured (Field Data)	m = 1.85	m = 2.
1.	3/18/60 1030	152	75.6	9.6	7.0	2.3 out	0.508	22.4	21.4	26
2.	3/18/60 1630	158	80.0	9.2	2.4	0.7 in	0.844	26.1	28.9	32
3.	3/19/60 1530	159	80.4	9.3	5.6	2.6 in	0.756	27.5	27.2	31
4.	3/19/60 2200	163	80.0	8.9	3.6	4.4 in	1.10	37.6	33.3	34
5.	4/2/60 1700	147	75.4	6.4	5.8	1.8 out	0.457	11.3[a]	11.3	11
6.	4/3/60 0300	156	82.3	6.0	1.9	2.6 in	1.13	16.2[a]	16.2	16
7.	6/13/60 1200	149	73.0	9.8	6.9	3.7 out	0.480	21.5	21.4	26
8.	6/13/60 1800	147	72.0	9.2	7.4	0.9 out	0.526	19.2	20.1	25
9.	6/14/60 0600	150	78.5	6.1	4.2	0.6 in	0.629	11.1	10.8	12
10.	6/14/60 2000	149	75.0	8.8	6.5	3.6 out	0.466	16.9	17.0	21

[a] Used to determine ϕ and n for "calculated" head losses. For m = 1.85, ϕ = 0.5 and n = 2.60. For m = 2.00, ϕ = 0.421 and n = 2.55.

of an 8-in. and two 6-in. boundary valves which had been inadvertently ope causing a leakage of about 15% of the High Service output to the Belmont Gr District.

Fig. 5 shows the future Torresdale High Service District with proposed ing improvements indicated by dashed lines. The existing (1960) distri also served by storage from Standpipes at A. It incorporates an addit area, called the Fox Chase Booster District, as indicated on the lower ed Fig. 5. This present composite service district is supplied not only fron but also from either the Oak Lane Pumping Station or the Fox Chase Bo Station via the Fox Chase District (both pumping stations are located belo

stem shown in Fig. 5). Random data for the existing district, with the Fox ase Booster shut down, taken from the log sheets of the Load Control Center pear in Table 6. Elevation of the station discharge gage at P_T is 0 ft. The essure gage at Point Y is located on a 12-in. grid main in the vicinity of int B in Fig. 5. Elevation of the field pressure gage at Point Y is 154 ft. e values from Items 5 and 6 were substituted in Eq. 2. Simultaneous solu-n gave ϕ = 0.542 and n = 2.60 for Point Y, for m = 1.85; ϕ = 0.421 and n .55 for m = 2.00. Comparison of the "calculated" versus the "measured" ad losses from P_T to Point Y when m = 1.85 is considered good and within ceptable limits of accuracy. Moreover, the P_T discharge pressure was only ainable to the nearest point. Load Control Center personnel stated that the ror in measurement from P_T to Point Y could be as much as 1.5 psi, depend- on operational circumstances. Torresdale High Service District is rela-ely residential in character with large wooded tracts and farm areas inter-ersed with localized concentrations of industry or institutional developments.

TABLE 7.—HAZEN-WILLIAMS C - VALUES - EXISTING DISTRICTS

Diameter, inches	C	Diameter, inches	C
Belmont High Service (Existing Arterial Piping in Fig. 6A)			
24	127	20	136
24	97	20	136
24	97	20	50
20	125	12	65
20	126	12	60
20	126	12	42
Torresdale High Service (Existing Arterial Piping in Fig. 5[a]			
24	146	16	117
24	138	16	117
24	135	16	106
24	132	16	70
24	128	16	70
24	104	16	48
24	79	12	118
20	115	12	83
20	114	12	77
20	110	12	67
16	137	12	67

[a] In Fox Chase District: 20 in., C = 124; 16 in., C = 48; 16 in., C = 44; 12 in., C = 47.

re, in this district, utilizing equalizing storage, the limitations of propor-nal loading are apparently applicable as well as the assumption that the pipes the average fit m = 1.85.

The use of the author's generalized network head loss characteristics in njunction with field data has been demonstrated in this discussion. The equa-ns under the limitations of assumed proportional loading and the application m = 1.85 for "new" and "old" pipes regardless of their condition give "cal-ated" head loss values in fairly good agreement to actual "measured" values

for the two Philadelphia districts cited. In the case of the Belmont High S vice District, use of m = 2.00 gives "calculated" values that compare as fa ably with the "measured" values as those obtained when using m = 1.8 = 1.9 would provide the best fit between actual and calculated values. It sh be noted that the gage at Point X is at the extreme end of the Belmont Service District and is located in a weak, local distribution grid which w be subject to more severe fluctuation than a point nearer the arterial pip For the Torresdale High Service District it was found that m = 1.85 prov the best fit as opposed to any other value for m. In the writers' opinion use of m = 1.85 for network analyses is reasonable and consistent when no design assumptions and overall accuracy of data and results is conside In Table 7 are listed the Hazen-Williams C-values for the Belmont High S vice and Torresdale High Service Districts. With such wide ranges o values, it appears that m = 1.85 for "new" and "old" pipes is thoroughly ju fied.[20]

For flow to storage the author states that $Q_p/Q_d > 1$. Obviously this alw holds true only when a district with equalizing storage has only one pum station as illustrated in Fig. 5 and by Table 1. In districts with equali storage and multiple pumping stations $Q_p/Q_d > 1$ only when the storage in rate is greater than the combined output of the other pumping stations Items 4 and 6 in Table 6 and the No. 4 series of runs in Table 4).

The foregoing suggests the use of the author's concepts for operational poses. High and low "normal" operating pressure values could be determ for critical areas to obtain criteria for activating warning devices. Co centers could establish valid operating standards which would result in r efficient and economical operation. In complex systems using two or r sources of input plus equalizing storage, more effective utilization of the tions to maintain adequate district pressures would be assured regardles tank inflow or outflow rates. Conversely, improper station operation or ir ularities in the distribution system would also be indicated.

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[20] Discussion of Walter L. Moore's "Relationships Between Pipe Resistance F ulas," Proceedings, ASCE, March, 1959, by M. B. McPherson, Proceedings, A September, 1959, pp. 143-155.

THE FOURTH ROOT n-f DIAGRAM[a]

Discussion by Donald Van Sickle

DONALD VAN SICKLE,[23] A.M. ASCE.—There has been presented a "gen-l resistance diagram" for use with the Darcy-Weisbach formula (and with Manning formula as well), presumably as a substitute for the much-roduced, but apparently seldom-used, Moody diagram. The merits and de-rits of the "general resistance diagram," as opposed to the common em-ical formulae, have been discussed frequently in the various technical jour-s.[24,25,26]

Since Mr. Blench is presenting the diagram as a practical aid and has pur-ely avoided detailed theoretical explanations, further discussion of the theo-ical points is beyond the scope of the paper. There are, however, a number eatures of the diagram which merit some discussion.

First, the proposed diagram incorporates the Blasius equation to represent nooth-turbulent" flow conditions, rather than the familiar von Kármán-ndtl logarithmic relationship of the Moody diagram. Basis for this change he belief that the logarithmic formula lacks a "completely logical founda-n and verification in the range of practical data." The writer feels, how-r, that most of the published data appear to substantiate the von Kármán-ndtl curve. The data presented by Peter Anton Lamont,[27,28] with one ex-tion, give considerable support to the von Kármán-Prandtl curve, and the eriments on new aluminum pipe,[29] on a large, concrete-lined tunnel,[30] and coated steel penstocks,[31] offer further evidence that the Blasius equation not applicable "in the range of practical data," except at Reynolds numbers

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a January, 1960, by T. Blench.

23 Hydr. Engr., Turner and Collie Cons. Engrs., Inc., Houston, Tex.

24 "Flow in Rough Conduits," by Henry M. Morris, Jr., Transactions, ASCE, Vol. , 1955, p. 373.

25 "Design Methods for Flow in Rough Conduits," Proceedings, ASCE, Vol. 85, July, 9, p. 43.

26 "Relationships Between Pipe Resistance Formulas," by Walter L. Moore, Pro-dings, ASCE, Vol. 85, March, 1959, p. 25.

27 "A Review of Pipe-Friction Data and Formulae, With a Proposed Set of Exponent-Formulae Based on the Theory of Roughness," by Peter Anton Lamont, Proceedings, he Inst. of Civ. Engrs., Part III, Vol. 3, April, 1954, p. 248.

28 "Formulae for Pipe-Line Calculations—Technical Committee Report B," Pro-dings, of the Third Congress of the Internal. Water Supply Assn., London, 1955.

29 "The Prediction of Flow Rates in Aluminum Pipe, Irrigation Tubing, and Fittings, its Hydraulic Efficiency After Years of Service," by J. S. Campbell and A. Brebner, nsactions, of the Engrg. Inst. of Canada, Vol. 3, No. 1, April, 1959, p. 43.

30 "Head-Loss Coefficients for Niagara Water Supply Tunnels," by J. B. Bryce and . Walker, The Engineering Journal, Engrg. Inst. of Canada, Vol. 42, No. 7, July, 9, p. 68.

31 "High Velocity Tests in a Penstock," by Maxwell F. Burke, Transactions ASCE, . 120, 1955, p. 863.

below 2 x 10^5. Above this limit, use of the Blasius equation will give f-va and friction losses lower than those which have been determined by exp ment.

Second, Mr. Blench defines his "roughness height" in terms of a base w is different from the base for k in the Moody diagram. Most of the publi roughness-height values have been expressed in terms of the "equivalent grain diameter" of Nikuradse. It would seem as though any departure f this established practice would cause considerable confusion and can be ju fied only under extraordinary circumstances. The reason given, howeve to "give e-values that suggest something like the over-all height of irregul ties in unlined rock tunnels . . . in deference to . . . the preference . . values that suggest visual roughness heights."

This attempt to give some physical significance to the e-value is, of cou commendable. Unfortunately, the practical value of such a move is quest able since, as the author notes, the irregularities in unlined rock tunnels "the only practical roughnesses that are fairly common and are really ass able by eye." For lined tunnels and all types of pipe conduits, then, the posed e-values will have no more physical significance than those on whicl Moody diagram is based.

It should be noted that the author "would be content to treat e as a number to be attached to roughness types that would be recognized intuitiv Such a code number has, apparently, not prevented the Hazen-Williams Manning equations from gaining widespread acceptance. Perhaps Mr. Bl would be willing to reconsider this change in the definition of e to prevent fusion.

Third, the proposed diagram eliminates the family of transition curves tween the "smooth turbulent" and "rough turbulent" flow regions which characteristic feature of the Moody diagram, and replaces them with a g of curves that represent the trends for different materials. The reasons g for this change are that the family of curves represents extrapolation wel yond the range of Colebrook's data, and that the transition curves are "to unlike ones that can be found for other materials." A comparison of Fi and 2 reveals that Colebrook's data covered a very considerable portion c transition region. Of the twenty transition curves shown in Fig. 1, onl upper two and the lower three seem to represent a significant extrapola Since there is a paucity of data in both of these areas, it is difficult to a the validity of these extrapolations, particularly in view of the abundan data corroborating the curves in the middle portion of the transition re

Presumably, the curves mentioned for "other materials" are those Nikuradse's experiments with uniform sand roughness,[32] and from the p by G. H. Hickox, A. J. Peterka, and R. A. Elder[33] with discussion, on steel and concrete lined tunnels. As noted, these curves are unlike the M diagram transition curves.

It is implied that the Colebrook transition curves should be used onl "commercial iron pipes," and the other curves used for concrete pip large tunnels. Lamont's data,[27,28] however, show asbestos-cement cement-lined pipe, and concrete pipe conforming to Colebrook's trans curves. In addition, there does not appear to be any unanimity of opinion i literature that concrete surfaces are comparable to the Nikuradse uni

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32 Engineering Hydraulics, by Hunter Rouse, John Wiley and Sons, Inc., New 1950, p. 112.

33 "Friction Coefficients in a Large Tunnel," by G. H. Hickox, A. J. Peterka, A. Elder, Transactions, ASCE, Vol. 113, 1948, p. 1027.

nd surfaces. To the writer's knowledge, there are no reliable data available concrete surfaces which indicate that the Nikuradse transition curves are ly representative of these surfaces, and he must conclude that the Nikuradse rves have been included simply to illustrate that some types of surface can- be represented by the Colebrook transitions.

It is pointed out that the Hickox, Peterka, and Elder curves (and presumably Ontario tunnel curve at the bottom of Fig. 2) "show the kind of trend to be pected from the N curves, but to a different degree." Hickox, Peterka, and der in their discussion of these data conclude, however, that "although the alachia tests were carefully made, they do not conform to either the kuradse or the Colebrook type of transition." Cyril Frank Colebrook,[34] in dition, in his discussion of the Ontario tunnel tests, comments that "despite appreciable experimental scatter, the test results are in very satisfactory reement with" the von Kármán-Prandtl smooth-pipe law.

The apparently conflicting interpretations of the meaning of these curves d the writer to conclude that it would be less advisable to extrapolate from m to establish a curve for a particular conduit than to extrapolate from the lebrook transition curves, for which a fairly large body of data is available. rtainly, before one could reasonably extrapolate from these new transition rves, it would need to be shown that they are truly representative of the ilable data on large conduits and tunnels.

An examination of the twenty-nine curves shown in Fig. 19 of Hickox, terka, and Elder's paper (not all of which are for large diameter conduits) icates that about half of the large conduit curves have the same trend as the lebrook transition curves, several are similar to the Nikuradse transition rve, and the six selected by Mr. Blench represent the most extreme depar- e from the Colebrook transition curve. It would seem to be difficult to de- ribe them as being "representative."

Colebrook has also presented additional data for lined, unlined, and partly- ed rock tunnels.[35] In the discussion of this paper[36] are presented additional rves for large diameter conduits. The general trend of the curves, with the ception of the Apalachia results, follows the Colebrook transition curves, hough Colebrook cautions that "the experimental scatter is generally too eat . . . to determine the actual shape of the transition curve for concrete rfaces." The data for the new Niagara tunnel[30] fall in a pattern somewhat nilar to the curve for the Ontario tunnel shown in the author's diagram. B. Bryce and R. A. Walker, however, make no attempt to relate these data either the Colebrook or Nikuradse transitions. It would appear appropriate Mr. Blench to include a number of these additional transition curves to ke it clear that the data for tunnels and large conduits are not all as differ- from the Colebrook transition as his initial selection would suggest.

Finally, the proposed transition zone curves are said to be "widely spaced cause the practical user is expected to sketch transitions in terms of (a) the en ones, (b) all other available information relevant to the specific problem, d (c) personal judgment." The wide spacing and completely different shapes

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34 "Turbulent Flow in Pipes with Particular Reference to the Transition Region Be- en the Smooth and Rough Pipe Laws," by Cyril Frank Colebrook, Journal, Inst. of . Engrs., Vol. 11, No. 4, February, 1939, p. 133.

35 "The Flow of Water in Unlined, Lined, and Partly Lined Rock Tunnels," Proceed- s, of the Inst. of Civ. Engrs., Vol. 11, September, 1958, p. 103.

36 Discussion of "The Flow of Water in Unlined, Lined, and Partly Lined Rock Tun- s," Proceedings, of the Inst. of Civ. Engrs., Vol. 12, April, 1959, p. 523.

of the curves, together with the usual lack of specific roughness informa available in designing a new conduit suggest that "personal judgment" w be a critical factor in the use of Fig. 2. Since most designers have not Mr. Blench's very considerable experience in conduit design, this import of the judgment factor would seem to limit the usefulness of the diagram. use of additional representative transition curves would increase the diagra value to less-experienced designers.

Of the objections to the use of the Moody diagram listed in Appendix 1, writer questions the validity of items i, ii, and iv, but must agree with it iii, v, and vi. In connection with item iii, however, Peter Ackers[37,38] published a set of design charts which considerably simplify the use of Moody diagram. It should be pointed out, however, that the charts are ba on the Colebrook transition curve and would not be applicable for diffe shapes of transition curves.

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37 "Resistance of Fluids Flowing in Channels and Pipes," by Peter Ackers, H Research Paper No. 1, Her Majesty's Stationery Office, London, 1958.

38 "Charts for the Hydraulic Design of Channels and Pipes," Hydr. Research P No. 2, Her Majesty's Stationery Office, London, 1958.

HYDROLOGIC STUDIES BY ELECTRONIC COMPUTERS IN TVA[a]

Discussion by J. L. Kovner

J. L. KOVNER.[6]—The use of automatic electronic computers in processing d analyzing hydrologic data is increasing at a rapid rate. Mr. Snyder has own one cost comparison between hand and machine methods which illus-tes why this development is taking place. In addition, it is pointed out that sk calculators have definite limitations in handling multiple regressions th ten independent variables.

The author mentions other basic computations such as mean daily discharge d sediment concentrations. These are not complicated calculations but the lume of data favors electronic computers. The USGS has a new water-level corder which punches head measurement at regular time intervals on paper pe and translates them to magnetic tape for immediate automatic processing.

The general application of the machines in solving engineering and statistical oblems makes it possible for TVA to have its own installation. Those not fortunate can purchase machine time at normal rates from computer labo-tories. No one engaged in extensive hydrologic investigations need forego e computing improvements described by Mr. Snyder.

The writer spent some time on the four watersheds prior to 1955, in the rly stages of the experiment. The design represented quite a departure om the standard practice of using control watersheds. It was apparent, then, at the four watersheds would have to be calibrated to account for varying eteorological conditions since the 4 by 4 Latin square design would not ef-ctively control these variables by the process of randomizing the treatments time. The writer was skeptical then that this could be done even though it peared that the small watersheds with uniform cover should behave almost e runoff plots. Mr. Snyder admits that the old techniques did not work and may be said that the high speed computing machines are the modus operandi the current state of affairs.

The author is certainly to be congratulated in developing this method of alysis. Although the final relationship for storm peaks is expressed explic-y in Eq. 5, one should not overlook the manner in which various hydrologic ocesses are taken into account, using a complicated bookkeeping system. is represents definite progress in hydrologic analysis and further applica-n of the techniques is warranted.

It appears, however, that the paper has some shortcomings in evaluating e results. The experiment was not set up to test whether the cultural treat-ents would have any effect. There was enough evidence on hand to verify that west North Carolina row crops on steep slopes would produce much higher

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[a] February, 1960, by Willard M. Snyder.

[6] Statistician, Rocky Mountain Forest and Range Experiment Sta., Forest Service, S. Dept. of Agric., Fort Collins, Colo.

storm peaks than pasture. The experiment could not demonstrate agai these facts even on the basis of low precision. The analysis must supply formation on the difference between treatments within fairly narrow confide limits.

In this respect, the results in Table 2 are not as good as indicated, nam that differences between peaks for various covers appear to be real; this is enough information. With standard errors of 50% and higher—even for past cover—it will require great improvement in the regressions to obtain satisf tory confidence limits. For watershed No. 1, pasture, the 95% confide limits are approximately -0.07 to 0.93 cfs. The predicted peaks used dep considerably from the means in Table 1, and this accounts for the large st dard errors. These predicted values are most interesting, however, si from a land-use point of view, it is the large storm peaks that are import

In this connection, the use of the regressions for tests of significance confidence intervals as suggested is open to some question. The assump of normality of the dependent variable for a net of fixed values of the indepe ent variables would have to be verified. Storm peak data are usually sign cantly skewed from normal. It also appears from Table 1 that variances crease with size of peak.

It is suggested, under the heading "Equation for Peak Discharge," that watershed 5 special analysis will be needed to adjust for changes in cover is evident that the regressions for corn and wheat on any watershed will h larger variances than pasture because of changes in cover from bare soi planting time to partial plant cover at full establishment.

In effect, separate regressions computed for the storms in different mo would have different parameters in Eq. 5, and a combined regression fo storms may not be justified. The analysis could probably be improved by c paring regressions on a seasonal or monthly basis.

Mr. Snyder uses an arbitrary and fixed time breakdown of the storm per There are four periods for storm peak hydrograph analysis and five per for storm runoff. These were evidently adopted on the basis of an ove analysis of runoff-producing storms. It would be interesting to know whe further subdivision of the storm duration into contributing periods would prove the relationships. The ultimate in electronic computing for this typ analysis appears to be a continuous flow of input data of the hydrologic meteorological variables.

SCOUR AT BRIDGE CROSSINGS[a]

Discussion by Joseph N. Bradley

JOSEPH N. BRADLEY,[13] M. ASCE.—The model results on scour at bridge tments and piers have been treated in a logical and commendable manner. e consistency with which the model results plot could be misleading howr, by making the prediction of scour appear as a simple routine procedure. ce Mr. Laursen has said little with regard to the limitations of his results, ew remarks on this phase of the subject may be appropriate.

Figs. 5 and 6 represent the results of model studies on abutment scour de under essentially ideal conditions, for example, with bed of granular maial free to move, a rectangular flow cross section, and a uniform velocity tribution. One can compare the author's results with those of a completely ependent set of experiments made with different size and gradation of bed terial, a constant depth of flow and a uniform velocity distribution, permed at Colorado State University,[8] and find that the two are in close agreent. This indicates that the model results are consistent and easy to duplie.

Where streams meet similar specifications in nature as those of the models, re should be a reasonable correspondence between model and prototype. ere are streams in India and Pakistan which do approach these so-called al conditions; the stream beds present an unlimited depth of alluvial maial, the river channels are extremely wide with more or less constant widthdepth ratio, and the velocities are low due to an unusually flat gradient (the rage is 2 ft in 4 miles in East Pakistan). Under such conditions the velocity tribution cannot vary greatly across the stream. Field measurements of ur at bridge abutments, spurs, and guide banks for the rivers of India and kistan are on record[14] and these show surprisingly good correlation with model results.[15]

Limited experience with scour on rivers in the United States show less farable comparisons. The reason is obvious; the gradients are steeper, the oss sections are irregular, the velocities are higher, and the velocity disbutions are far from uniform. Under these conditions the greatest scour es not necessarily occur at the abutments but is more likely to be found in

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a February, 1960, by Emmett M. Laursen.

13 Hydr. Engr., Internatl. Engrg. Co., Inc., Dacca, East Pakistan.

8 "Backwater Effects of Bridge Piers and Abutments," by H. K. Liu, J. N. Bradley, E.O. Plate, Civ. Engrg. Sect. Proj. Report CER., 57HKL2, Colorado A. and M. Cole, Fort Collins, Colo., 1957.

14 "The Behavior and Control of Rivers and Canals," by Sir Claude Inglis, Research lication 13, Part II, Central Water Power Irrigation and Navigation Report, Poona search Sta., 1949.

15 "Field Verification of Model Tests on Flow Through Highway Bridges and Culverts," Carl F. Izzard and Joseph N. Bradley, Proceedings of the Seventh Hydr. Conf., Iowa, 8, pp. 225-243.

the portion of the channel where the depth of flow and velocity are greate Records of the United States Geological Survey of bridge sites in Mississip where the beds are generally of alluvial material, show this to be true. Th is also evidence from past floods in various sections of the United States t the settlement of piers in the deeper portion of the channel is more comn than abutment failures due to scour. Yet if Figs. 3 and 6 are consulted, order, it is found that prediction from the model gives scour depth up to th times the pier width for the center of the channel, while scour up to six tin the depth of flow is supposedly possible at abutments. The latter can resul fantastic figures, which are true in the case of the model with rectangu cross section, where all scour is concentrated at the abutments; but such p dictions are unreasonable when applied to irregular cross sections in the fie

This discussion was not written to confuse the issue or discredit the mo results (which are valid for the conditions tested). Rather it is to point some of the remaining unknowns and (1) encourage investigators to mak concerted effort to take soundings of streams at constrictions both before during floods for the purpose of better understanding the field problem making better utilization of the model results; and (2) to warn engineers to use the model information blindly but to treat each river crossing as an in vidual problem, using the model results as a guide rather than a definite sc tion. Returning to item (1), further model studies will not produce the ansv desired; field measurements are the only alternative. A further comment item (2) is that model results applied to abutment scour in the United Sta will certainly fall on the side of safety. The extra cost of unduly deep footi may, therefore, be sufficient to finance a comprehensive field study in a v short time.

AP EFFICIENCY OF RESERVOIRS, DEBRIS BASINS, AND DEBRIS DAMS[a]

Discussion by Herman G. Heinemann

HERMAN G. HEINEMANN,[17] F. ASCE.—This paper is a very helpful sum-
ry of the better-known articles on the subject of trap efficiency. It also
ps call attention to the need for more study, research, and analysis on this
remely important matter.

Speaking generally, there are three sedimentation considerations of impor-
e that face the planner or designer of a proposed reservoir: (1) How much
iment will be delivered to the proposed reservoir (sediment yield)? (2)
t percentage of the delivered sediment will be retained in the reservoir
p efficiency)? (3) Where will the retained sediment be deposited (sediment
ribution)? Help on any of these questions is a definite contribution to the
wledge of reservoir sedimentation.

t is assumed that the definition of trap efficiency given in the "Introduction"
not intended. Trap efficiency is generally expressed as the ratio between
iment accumulation and sediment inflow.

For clarity purposes, it should also be pointed out that the term "sediment
duction" as it is used under the heading "Summary of Paper by Brune" and
in the referenced article by Glymph, has been largely replaced in the
e recent years by "sediment yield." This also pertains to the heading of
7 in Table 1. This is made apparent by the use of the term "sediment
d" in the later portions of this paper where reference is not made to arti-
or papers of a number of years ago. These terms are still used somewhat
rchangeably, but sediment production is now considered as referring to
sion or sediment volume from small areas—plots—single cropped, etc.,
reas sediment yield refers to the sediment volume from more complex
ersheds or subwatersheds—as defined by Glymph.[18]

Since the trap efficiency for a structure is a changing value, high at first
lower as the reservoir becomes filled with sediment, it is the writer's
ion that a date or a period of time should always accompany the trap-
ciency value given for any reservoir; it was expressed in this way for a
ber of reservoirs included in the paper. One should not give a trap effi-
cy value for "the life of a structure" without defining this or giving the
od of years considered as the useful or design life. Trap-efficiency values
be more meaningful if expressed as follows:

1) "The 50 yr. trap efficiency of x reservoir is estimated at Y%."
2) "The trap efficiency of x reservoir is Y% (1959)."

February, 1960, by Charles M. Moore, Walter J. Wood and Graham W. Renfro.
[17] Hydr. Engr., Sedimentation Studies, Watershed Tech. Research Branch, Soil and
r Conservation Research Div., Agric. Research Service, Lincoln, Nebr.
[18] "Studies of sediment yields from watersheds," by Louis M. Glymp, Jr., Internatl.
n of Geodesy and Geophysics, Tenth Genl. Assembly, Rome, Italy, 1954, Vol. 1, pp.
191, 1955, illus.

TOLKMITT'S BACKWATER AND DRAWDOWN CURVE TABLES[a]

Discussion by Ven Te Chow

VEN TE CHOW,[19] F. ASCE.—Tolkmitt developed his tables on the assump-
n of a parabolic channel having the width sufficiently large compared with
depth y, so that the wetted perimeter P can be considered approximately
al to the top width T. Accordingly, T varies with $y^{0.5}$; the water area A
ies with $y^{1.5}$; and the hydraulic depth D and hydraulic radius R both vary
h y. Thus, for Tolkmitt's special case, the hydraulic exponents[20] can be
wn as N = M = 4.

For channel sections of a general shape, the flow-profile equation[20] is

$$x = \frac{y_n}{S_o}\left[u - \int_0^u \frac{du}{1 - u^N} + \left(\frac{y_c}{y_n}\right)^N \int_0^u \frac{u^{N-M}}{1 - u^N}\,du\right] + \text{constant} \quad (8)$$

ere, using author's notation, y_n = d, S_o = s, u = (d + z)/d and (d + h)/d for a
sing of water level, and u = (d - z)/d and (d - h)/d for a lowering of water
el. When Chezy's formula is used, it can be shown that $y_c/y_n = sC^2/g^{\,1/4}$.
us, Eq. 8 can be written partially in author's notation as

$$x = \frac{d}{s}\left[u - \int_0^u \frac{du}{1 - u^N} + \left(\frac{sC^2}{g}\right)^{N/4} \int_0^u \frac{u^{N-M}}{1 - u^N}\,du\right] + \text{constant} \quad (9)$$

ere $\int_0^u \frac{du}{1 - u^N}$ = F (u, N), a varied-flow function. For Tolkmitt's special
e, Eq. 9 is reduced to

$$x = \frac{d}{s}\left[u - \left(1 - \frac{s\,C^2}{g}\right)\int_0^u \frac{du}{1 - u^4}\right] + \text{constant} \quad (10)$$

ere

$$\int_0^u \frac{du}{1 - u^4} = F(u, 4) = \frac{1}{8}\ln\left(\frac{u + 1}{u - 1}\right)^2 + \frac{1}{2}\tan^{-1} u \quad (11)$$

ay be noted that F (u, 4) = $\pi/4$ when u becomes infinity.

For a raising of water level, X = u = (d + z)/d = 1 + z/d and u > 1. Thus,
11 becomes

$$F(u, 4) = \frac{1}{4}\ln\left(1 + 2\frac{d}{z}\right) + \frac{1}{2}\tan^{-1}\left(1 + \frac{z}{d}\right) \quad (12)$$

May, 1960, by R. D. Goodrich.

9 Prof. of Hydr. Engrg. in charge, Univ. of Illinois, Urbana, Ill.

0 "Open-Channel Hydraulics," by Ven Te Chow, McGraw-Hill Book Co., Inc., New
, 1959.

From Eq. 2

$$f(X) = u - F(u, 4) + \frac{\pi}{4} \ldots\ldots\ldots\ldots\ldots$$

In the varied-flow function table,[20] for $u > 1$, a constant, $\pi/4$, was subtr ed from F (u, 4) to make the function value near to zero when the varia becomes very large. Let F (u, 4) represent the table value. Then,

$$f(X) = u - F(u, 4) \ldots\ldots\ldots\ldots\ldots$$

This relation indicates that the Tolkmitt's table can be replaced by the ge varied-flow function table for flow-profile-computation purposes. For ample, with u = 1.060, the table value[20] for F (u, 4) for N = 4.0 is 0.506 Eq. 14, f(X) = 1.060 - 0.506 = 0.554. The above relation also applies to = (d + h)/d = 1 + h/d.

Similarly, the varied-flow function for a lowering of water level, with = (d - z)/d = 1 - z/d and $u < 1$, can be shown as

$$F(u, 4) = \frac{1}{4} \ln\left(2\frac{d}{z} - 1\right) + \frac{1}{2} \tan^{-1}\left(1 - \frac{z}{d}\right) \ldots\ldots$$

From Eq. 4,

$$f(X) = F(u,4) \ldots\ldots\ldots\ldots\ldots$$

Thus, Tolkmitt's table value is identical with the value given by the va flow function table. This also applied to X = u = (d - h)/d = 1 - h/d.

Tolkmitt's table is widely known in European literature. The author s be commended for introducing this table to the American literature.

HOOD INLET FOR CLOSED CONDUIT SPILLWAYS[a]

Discussion by C. D. Smith

. D. SMITH,[10] F. ASCE.—It is heartily agreed that the hydraulics of culs is not as simple as was once believed. In this regard, Mr. Blaisdell's r represents a further useful contribution to this interesting and important ect.

ne feature of the hood inlet which was not mentioned, and which should ree attention, is a discussion of the limitations of the design. In the writer's on, there are very definite structural and hydraulic limitations which ld be considered by the designer when he is making his selection of inlet gn.

he first point concerns the design of the splitter-type anti-vortex wall. In , a design using the hood inlet was carried out under the direction of the er for a 60-in. corrugated iron pipe. At that time, the only data reported he anti-vortex device was for the splitter-type wall. According to the mum dimensions, the splitter wall would have to cantilever upstream 5 ft beyond the invert of the pipe. It was found that several ring stiffeners d be required for the pipe, and structural steel angles would be required inforce the thin splitter wall. The design was complicated, and there was some question as to the adequacy of the splitter wall to resist impact due oating ice cakes or debris. Finally the design was abandoned in favor of re conventional reinforced concrete inlet with a rounded lip.

may be that problems similar to this were the reason for the objections ed to the splitter type anti-vortex wall. The anti-vortex plate of Fig. 8(f) ated to be equally efficient. It appears to be much simpler structurally, hould be a satisfactory replacement for the wall.

ne second point concerns the limitations of any design which gives slug Slug flow is unsteady flow and associated with such flow are varying sures, noise, vibration, and pounding at the inlet, and intensified wave act the outlet. It appears that acceptability or rejection of these flow conns depends on the size of the works. For small stock-watering dams usn 18 in. or 24 in. pipe these problems are probably not critical. For large of 60 in. or greater it would seem a wise precaution to avoid designs n give slug flow.

should be mentioned that when the spillway operates, slug flow would prenost of the time. This is because the inlet primes with a fairly low dise, but the pipe does not flow full continuously until this discharge has exceeded several times.

e final and most important point concerns the development of subspheric pressures in the inlet. It was stated under the heading "Theory"

Iay, 1960, by Fred W. Blaisdell.

Assoc. Prof. of Civ. Engrg., Univ. of Saskatchewan, Saskatoon, Canada.

that if the pipe is steep, the pressure "will ordinarily be sub-atmosph throughout most of the conduit length." This would also be true for any o type of inlet on a steep pipe flowing full. However, the local contractio flow in the hood inlet will give a pressure much lower at that point than a where in the pipe.

Consider a sharp edged re-entrant tube flowing full: The coefficient of jet contraction C_c = 0.5, and the coefficient of head loss

$$K_e = \left(\frac{1}{C_c} - 1\right)^2 = 1.0.$$

The pressure head at the invert at the inlet will be

$$\frac{P}{w} = H - \frac{V^2}{2g} \quad \ldots\ldots\ldots\ldots\ldots\ldots$$

where V is the local velocity of the contracted jet, and not the average vel in the pipe. Therefore,

$$\frac{P}{w} = H - \frac{Q^2}{C_c^2\, A^2\, 2g} \quad \ldots\ldots\ldots\ldots$$

With C_c = 0.5, the local velocity head will be 4 times as great as the ve ty head based on average pipe velocity, and the pressure head will be co spondingly lower. Eq. 16 has been verified experimentally by the write others for a sharp edged re-entrant tube flowing full.

Since the value of K_e for the hood inlet is also close to unity, one migh pect that C_c would also equal 0.5. Actually, the cases are not exactly parable, as the bevel cut on the hood inlet has some modifying influence. not possible to determine C_c for the hood inlet flowing as an orifice, bec it will not flow as an orifice. However, the "effective" value of C_c can be culated from Eq. 16 if Q, A, H, and P/w are measured.

In tests conducted by the writer, piezometer readings were taken alon pipe invert downstream from the hood inlet. The pressures were not a for the hood inlet as for the sharp edged re-entrant. It was calculated tha effective value of C_c was approximately 0.6. This is a higher coefficient for the ordinary sharp-edged re-entrant, but it appears reasonable fo case.

It was also observed that the crown pressures along the hood inlet greater than 1 diameter above the invert pressures. This also appears sonable when the upward component of velocity, directed toward the cro the inlet, is considered. Actually, the flow pattern is exceedingly com and the value of C_c = 0.6 is suggested only as a value for computing press rather than giving an actual area of flow in the inlet.

Eq. 16 may be written in dimensionless terms by dividing by D

$$\frac{P/w}{D} = \frac{H}{D} - \frac{\left(\frac{Q}{C_c\, A}\right)^2}{2\,g\,D} \quad \ldots\ldots\ldots\ldots$$

Substitution of C_c = 0.6, and reduction, yields an equation in the same as Fig. 6:

$$\frac{P/w}{D} = \frac{H}{D} - 0.07\left(\frac{Q}{D^{5/2}}\right)^2 \quad \ldots\ldots\ldots\ldots$$

illustrate the significance of Eq. 18, suppose a prototype pipe 48 in. in
eter were to be built with such a length, slope, and roughness as to yield
ischarge curve shown in Fig. 6 for a pipe slope of 0.200. This pipe should
ull when $H/D = 2$ and give a discharge corresponding to $Q/D^{5/2} = 15.5$.
itution of these values in Eq. 18 shows that the pressure head at the invert
l be -14.8 pipe diameters. This would correspond to -59.2 ft on the pro-
e, so obviously it would not perform as indicated on the model under these
tions.

alue of -1/2 an atmosphere is often imposed as a design limit in hydrau-
ructures. Lower pressures are considered to be in the realm of potential
ation. If this value was used in Eq. 18, then the permissible maximum
of pipe for the conditions of the above example would be 14 in.

the writer's view, the negative pressures at the inlet impose a very defi-
imit on the size of pipe for which the hood inlet may be used with success.
lesigner should check each particular case, using Eq. 18, to insure that
ures are within acceptable limits.

ERRATA

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Proceedings of the American Society of Civil Engineers

April, 1960

p. 95. Delete lines 5 and 6 of paragraph 3 and substitute therefore: had proved, the dissipation of energy balances nearly with the productior in the range of approximately $0 < y/h < 1/2$, and with the diffusion rate outer region $(0.8 < y/h \leq 1)$, and the turbulence

p. 95. Delete Eqs. 25 and 26 and replace with

$$\text{Dissipation} = A' \rho \nu \frac{\overline{u^2}}{\lambda^2} = A \rho \left(\frac{\sqrt{\overline{u^2}}}{\overline{U}_*} \right)^3 \frac{\overline{U}_*^3}{L} \quad \ldots\ldots\ldots$$

$$\text{Diffusion} = \frac{d}{dy} \overline{\left(\rho \mu \frac{u^2 + v^2 + w^2}{2} \right)} = A'_1 \frac{d}{dy} \left(\rho \frac{\overline{u^2 v}}{2} \right)$$

$$= A'_1 \frac{\rho U_*^3}{h} \frac{d}{d(y/h)} \left(\frac{\overline{u^2 v}}{\overline{U}_*^3} \right) \ldots\ldots\ldots\ldots$$

Considering the isotropic character of turbulence in the outer (open cha or center (pipe) region, Eq. 25' can be written as

$$\text{Diffusion} = A_1 \frac{\rho \overline{U}_*^3}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

Therefore, by equating Eq. 25 to Eq. 24 and Eq. 25'' we have

$$\frac{L}{h} = \left\{ \begin{array}{l} B \dfrac{y/h}{1 - y/h} \left(\dfrac{\sqrt{\overline{u^2}}}{\overline{U}_*} \right)^3 = f_1(y/h) \\ \qquad (0 < y/h < 0.5) \\ B' \left(\dfrac{\sqrt{\overline{u^2}}}{\overline{U}_*} \right)^3 = f'_1(y/h) \\ \qquad (0.8 < y/h \leq 1) \end{array} \right\} \ldots\ldots\ldots\ldots$$

p. 96. Just prior to the last paragraph insert

so, the diffusion coefficient for uniform open channel flows is given from 8) as

$$D_z = \text{const.} \sqrt{g}\ h^{3/2}\ Se^{1/2} \qquad (31)$$

1960

21. In Eq. 9, insert a parenthesis before the term 0.15 and after the term /2.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW) divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4), 2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2), 2142(CO2), 2143(CO2), 2144(HW3), 2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9), 2150(HY9), 2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3), 2159(IR3), 2160(IR3), 2161(SA5), 2162(SA5), 2163(ST7), 2164(ST7), 2165(SU1), 2166(SU1), 2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436(ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445(EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454(EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

MAY: 2460(AT1), 2461(ST5), 2462(AT1), 2463(AT1), 2464(CP1), 2465(CP1), 2466(AT1), 2467(AT1), 2468(SA3), 2469(HY5), 2470(ST5), 2471(SA3), 2472(SA3), 2473(ST5), 2474(SA3), 2475(ST5), 2476(SA3), 2477(ST5), 2478(HY5), 2479(SA3), 2480(ST5), 2481(SA3), 2482(CO2), 2483(CO2), 2484(HY5), 2485(HY5), 2486(AT1)[c], 2487(CP1)[c], 2488(CO2)[c], 2489(HY5)[c], 2490(SA3)[c], 2491(ST5)[c], 2492(CP1), 2493(CO2).

JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502(EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511(EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520(PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529(HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

JULY: 2541(ST7), 2542(ST7), 2543(SA4), 2544(ST7), 2545(ST7), 2546(HY7), 2547(ST7), 2548(SU2), 2549(SA4), 2550(SU2), 2551(HY7), 2552(ST7), 2553(SU2), 2554(SA4), 2555(ST7), 2556(SA4), 2557(SA4), 2558(SA4), 2559(ST7), 2560(SU2)[c], 2561(SA4)[c], 2562(HY7)[c], 2563(ST7)[c].

AUGUST: 2564(SM4), 2565(EM4), 2566(ST8), 2567(EM4), 2568(PO4), 2569(PO4), 2570(HY8), 2571(EM4), 2572(EM4), 2573(EM4), 2574(SM4), 2575(EM4), 2576(EM4), 2577(HY8), 2578(EM4), 2579(PO4), 2580(EM4), 2581(ST8), 2582(ST8), 2583(EM4)[c], 2584(PO4)[c], 2585(ST8)[c], 2586(SM4)[c], 2587(HY8)[c].

c. Discussion of several papers, grouped by divisions.